Effect of Waiting Time Before Re-vibration on Flexural Behavior of Reinforced Concrete Beam

A Thesis

Submitted to the Council of the College of Erbil Technical Engineering College at Erbil Polytechnic University in Partial Fulfillment of the Requirements for the Degree Master of Science in Civil Engineering

By

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عَنْ أَبِي هُرَيْرَةَ رضي الله تعالى عنه: أَنَّ رَسُولَ اللَّهِ ﷺ قَالَ
إِذَا مَاتَ ابنُ آدم انْقَطَعَ عَنْهُ عَمَلُهُ إِلَّا مِنْ ثَلَاثٍ: صَدَقَةٍ جَارِيَةٍ، أو عِلْمٍ يُنْتَفَعُ بِهِ، أَوْ وَلَدٍ صَالِحِ يَدْعُو لَهُ
رَوَاهُ مُسْلِمٌ

Abu Hurairah (May Allah be pleased with him) reported:

The Messenger of Allah (ﷺ) said, "When a man dies, his deeds come to an end except for three things: a permanent charity, a knowledge which is beneficial, or a virtuous descendant who prays for him (for the deceased).

DECLARATION

I declare that the Master Thesis entitled: "**Effect of Waiting Time Before Re-vibration on Flexural Behavior of Reinforced Concrete Beam"** is my own original work, and hereby I certify that unless stated, all work contained within this thesis / dissertation is my own independent research and has not been submitted for the award of any other degree at any institution, except where due acknowledgment is made in the text.

Signature:

Student Name: Ali Dilshad Nuraddin

Date: / / 2023

SUPERVISOR CERTIFICATE

This thesis has been written under my supervision and has been submitted for the award of the degree of Master of Science in Civil Engineering with my approval as supervisor.

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I confirm that all requirements have been fulfilled.

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Date: / / 2023

EXAMINING COMMITTEE CERTIFICATION

We certify that we have read this thesis: "**Effect of Waiting Time Before Re-vibration on Flexural Behavior of Reinforced Concrete Beam"** and as an examining committee examined the student (**Ali Dilshad Nuraddin**) in its content and what related to it. We approve that it meets the standards of a thesis in terms of scope and quality for the degree of Master in Civil Engineering.

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Dean of Erbil Technical Engineering College
Date: / / 2023

DEDICATION

This work is humbly dedicated to all my valuable treasures in life:

To My Mother

A strong and gentle soul who taught me to trust Allah, believe in hard work that so much could be done with little.

To My Father

For earning an honest living for us, for supporting and encouraging me to believe in myself.

To My wife

For her love, sacrificial care for our children and me which made it possible for me to complete this work.

To My Angel (Nil)

Who is indeed a treasure from God.

To My Three Brothers

Who they support, understanding and believing me, make me be proud for the works which I do in my life.

To My Teachers

Along with all hardworking and respect.

My friends who encourages and supports me

All the people in my life who touch my heart. I dedicate this research.

I am forever thankful

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In the name of Allah, Most Merciful and Most Gracious

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ABSTRACT

Re-vibration, which is the process of repeating the operation of vibration of fresh concrete after a time interval, may be useful to enhance the mechanical properties of concrete (compressive strength, tensile strength, flexural strength, and modulus of elasticity), and also to get the maximum applied load, first crack load, deflection at mid span, stiffness and strain, particularly when successive layers of fresh concrete were placed and the upper layer of fresh concrete was partially hardened. After a period of time, the aggregate particles are rearranged by the re-vibration process, and any trapped water is removed, potentially enhancing the concrete's compressive and tensile strengths. The use of re-vibration can help to remove plastic shrinkage cracks for exposed concrete. The amount of time that re-vibration lasts has a big impact.

Using the re-vibration technique in construction of structural members is expected to improve the structural properties of the beams, and cracking. The effect of the waiting time before re-vibration must be investigated and time duration of vibrations on the structural response of flexural reinforced concrete beams to establish and verify the best process to apply this technique.

The purpose of this study is to examine the impact of waiting time before re-vibration. and variable time duration for vibration and re-vibration operation of structural response for flexural reinforced concrete beams and using Ordinary Portland Cement-type 1, with w/c ratio of 0.4, and a greater number of re-vibration time lag intervals ranging from half an hour to two hours to evaluate the impact of re-vibration on the mechanical properties of concrete with different time duration of vibrations ranging from 15 to 60 seconds.

The experimental schedule includes a total of 28 twenty-eight rectangular reinforced concrete beams of dimension (125 mm x 250 mm) and length of 1500 mm was prepared for this work. Which were classified into seven groups Each group includes four samples. Beams for group (A, B, C, D, E and F) reinforced with longitudinal top reinforcement of (2ø10mm) and bottom reinforcement of (2ø12mm) with transvers reinforcement (Stirrups) ø12mm all over the beams except group F which are without transverse reinforcement and group G is without longitudinal and transverse reinforcement. Additionally, 144 cylinders with dimensions of 300 mm in height and 150 mm in diameter were utilized to evaluate the concrete's compressive and tensile strengths. After 56 days, these samples were analyzed in an attempt to study the impacts of vibration, and the effect of re-vibration delay and vibration duration on the development of concrete strength.

The results have shown that the mechanical properties of concrete, ultimate load, first crack load and deflection at mid-point with various time duration and re- vibration techniques was increased for the 1st one hour and re-vibrated for 45 seconds time duration. After that for waiting time 1.5 and 2 hours and re-vibrated for 60 seconds was decreased.

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LIST OF ABBREVIATIONS

Symbol	Meaning
ACI	American Concrete Institute
NRMCA	National Ready Mixed Concrete Association
TRC	Transportation Research Council
ASTM	American Society for Testing and Materials
CCANZ	Cement and Concrete Association of New Zealand
SANS	South African National Standard
ISS	Indian Standard Specification
$Ca(OH)_2$	Calcium Hydroxide
(**C-S-H**)	Calcium Silicate Hydrate
OPC	Ordinary Portland Cement
C_3S	Tri-calcium silicates
C_2S	Di-calcium silicates
CSIR	Council for Scientific and Industrial Research
H	Hydrogen
$C_3S_2H_3$	Calcium Silicate Hydrates
CH	Calcium hydroxide
C_3A	Tri-calcium aluminate
SiO_2	Silicon dioxide
Cao	Calcium oxide
AL_2O_3	Aluminium oxide
Fe_2O_2	Iron(II) oxide
MgO	Magnesium oxide
SO_3	Sulfur trioxide
C_3A	Tricalcium aluminate
L.S.F	Load Sharing Facility
C_4AF	Tetra calcium alumina ferrite
f_y	Tensile yielding stress of steel reinforcement (MPa)
f_u	Ultimate Strength of steel (MPa)
f'_c	Compressive strength of concrete (MPa)
f_{sp}	Splitting tensile strength (MPa)
P	Load (kN)
L	Total length of the beam (mm)
d	Effective depth of the beam (mm)
P_{cr}	Cracking load (kN)
P_u	Ultimate failure load (kN)

δu	Deflection at ultimate load (mm)
δ_{cr}	Deflection at cracking load (mm)
P_{cr}	Cracking load (kN)
k	Stiffness factor
V_c	Shear strength of concrete (N)
b_w	Width of the beam (mm)
A_v	Cross section area of stirrup legs ($2A_b$) (mm^2)
V_s	Shear strength of stirrups (N
S	Spacing between stirrups
V_u	Ultimate shear strength of the beam
λ	light weight concrete modification
σ	Standard deviation
R	Ratio
r	Correlation
Var	Variance
M_n	Nominal bending moment (kN.m)
P_n	Nominal load (kN)
a	Shear span (mm)
M_{cr}	Cracking moment (kN.m)
A_s	Area of reinforcement in tension zone (mm^2)
A_s'	Area of reinforcement in tension zone
Ig	Moment of inertia of the transformed section
f_r	Modulus of rupture of the concrete (MPa)
y_t	Distance from the neutral axes to the tension face (mm)
t_{rv}	Re-vibration time (sec)
t_w	Waiting time after initial vibration (min)
R_{avg}	Average ratio of R
R_{Max}	Maximum ratio of R
R_{Min}	Minimum ratio of R
$M_{n.c}$	Bending moment of control beam (kN.m)
$f'_{c.c}$	Compressive strength of control beam (MPa)
$f_{sp.c}$	Splitting tensile strength of control beam (MPa)
$P_{cr.c}$	Cracking load of control beam (kN)
$P_{u.c}$	Ultimate failure load of control beam (kN)
$\delta_{u.c}$	Deflection at ultimate load of control beam (mm)
h	Height of the beam (mm)

CHAPTER ONE

INTRODUCTION

1.1 Overview

Concrete is an artificial stone that is set in place when it is still flexible. Numerous issues can arise throughout the concrete-laying process that could compromise the hardened concrete's qualities. The goal of this study is to look at how vibration affects concrete's qualities, both in the fresh and hardened states, specifically the strength. The purpose of this experimental work is to investigate the influence of vibration time duration and delaying the re-vibration of concrete in order to gain a better knowledge of the effects and processes involved. The influence of the hot-dry climates of Kurdistan and Iraq, as well as the period of beneficial vibration, are especially targeted.

Concrete design has been considered for strength and durability performance for so many decades. Cracks are clearly a concern that affects the strength and durability of concrete. Although concrete cracks can arise as a result of various reasons such as weather conditions and concrete settlement, managing such cracks requires specific care in order to construct reliable and visually appealing concrete structures.

Early age deterioration of concrete is a chronic issue caused by rapid complex volume changes such as plastic and dry shrinkage. These volume variations produce tensile stresses in the material when its strength is still relatively low. The resulting strains may produce immediate cracking, resulting in premature deterioration and affecting the integrity, durability, and long-term service life of a concrete structure (ACI 2000).

Plastic shrinkage cracking, which forms on the surface of fresh concrete within the first few hours after the concrete is cast, is a common type of early age deterioration of concrete. In concrete structures exposed to a hot, humid, and windy environment, plastic shrinkage cracking can be a concern due to a quick loss of water from the top of concrete before it has set. These weather indicators help predict whether plastic shrinkage fractures will appear (NRMCA 1960).

The most significant component that influences the long-term durability of concrete is typically characterized as the crack width, which can be relatively large on the upper surface, ranging from 2 to 3 mm (0.08 to 0.12 in), but frequently reduces quickly below the surface. The following occurrences, such as drying shrinkage and loading, are likely to contribute to the spread of the plastic shrinkage cracks (TRC 2006).

There are many methods that have been shown to decrease the risk of plastic shrinkage cracking, but it is important to accurately quantify each method's performance before weighing the costs and advantages of each choice. Controlling the impacts of weather has been suggested as one way to stop early aging cracks (Snell 2008). Low concentrations of randomly arranged short fibers have also been investigated as a potential alternative method to prevent plastic from shrinking and splitting (Qi at el 2003).

ASTM C1579 is an ASTM test technique for quantifying the plastic shrinkage cracking behavior of concrete mixtures (ASTM 2013). This approach was standardized as a test method for evaluating constrained fiber reinforced concrete plastic shrinkage cracking. The consideration of using the same test method as a possible approach to evaluate the plastic shrinkage cracking of restrained reinforced concrete slabs exposed to internal vibration is reported as a result of the study of re-vibration technique of concrete as a method to reduce the plastic shrinkage cracking.

It has been discovered in particular that "re-vibration" of concrete can result in appreciable gains in strength as long as the concrete is once more returned to a plastic state. Several theories have been proposed to explain how re-vibration increases strengths. The experimental program described in this thesis, which examines how re-vibration affects the mechanical strength of concrete, is intended to aid in understanding the positive impacts that re-vibration introduces.

1.2 General about Concrete:

Concrete in general is a commonly used structural material and it consists essentially from mixing water, cement, aggregate and if necessary special additives with selected correct proportions of the ingredients to produce concrete mix with specific properties.

The chemical reaction of cement and water is relatively slow and needs several days to complete the reaction, therefore, the strength of concrete is not attained quickly after casting. Concrete should be cured to prompt the hydration of cement and to keep the concrete saturated or nearly saturated.

The concrete has to be satisfactory in its hardened state and also in its fresh state while being transported form the mixer and placed in the form-work. The requirements in the fresh state are that the consistence of the mix be such that it can be compacted by the means desired without excessive effort, and also that the mix be cohesive enough for the method of placing used, not to produce segregation with consequent lack of the finished product.

The usual primary requirement of the good concrete in its hardened state is satisfactory compressive strength, density, durability, tensile strength, impermeability… etc.

The properties of fresh concrete are important only in the first few hours of its history whereas the properties of hardened concrete assume an importance which is retained for the remainder of the life of concrete. The important properties of hardened concrete are strength, deformation under load, durability, permeability and shrinkage.

In general, strength is considered to be the most important property and quality of concrete is often judged by its strength. There are many occasions when other properties are more important, for example, low permeability and low shrinkage are required for water-retaining structures. Although in most cases an improvement in strength results in an improvement of the other properties of concrete with few exceptions. For example, increasing the cement content of a mix improves strength but results in higher shrinkage which in extreme cases can adversely affect durability and permeability.

Since the properties of concrete change with age and environment, it is not possible to attribute absolute values to any of them.

1.3 Compaction of concrete:

1.3.1 General:

During the early stage, when the concrete is still fresh, two qualities are especially important:

(1) Workability.

(2) Cohesiveness (influences the resistance of concrete to constituent segregation during transport, placement, and compaction).

Concrete's viscous drag and interference between the aggregate and cement particles are the main causes of flow resistance. Greater workability will result from the solids being pushed further apart by additional water. When the applied tension is high enough to overcome the resistance between the solid particles, plastic concrete will start to flow. Vibration typically overcomes the resistance.

A significant amount of air is entrapped during the manufacturing of concrete, and partial segregation may occur during transportation. If the entrapped air is not removed and the coarse aggregate segregation is not corrected, the concrete may be porous, non-homogeneous, and weak.

Compaction is the process of removing entrapped air and uniformly placing concrete from a homogeneous dense mass. It makes spreading the concrete in the forms difficult. The friction also keeps the concrete from making direct contact with the four reinforcements, resulting in a poor bond between the reinforcement and the surrounding concrete. Compaction aids in the reduction of frictional forces. It is also possible to reduce friction by adding more water than is required to hydrate the cement.

Compaction of concrete is the process of expelling entrapped air from concrete during the placing and mixing of concrete. To achieve full compaction and maximum density with reasonable compaction efforts available on site, a mix with adequate workability is required. It is also widely accepted that the mix should not be too wet for easy compaction, as this reduces the strength of the concrete. For concrete compaction, the following methods are used:

1.3.2 Compaction by Hand Tamping:

Hand compaction of concrete is adopted in case of unimportant concrete work of small magnitude. Hand compaction consists of rodding, ramming or tamping.

When hand compaction is adopted, the consistency of concrete is maintained at a high level. The thickness of the layer of concrete is limited to about 15 to 20 cm. Rodding is nothing but poking the concrete with about 2 m long, 16 mm diameter rod to pack the concrete between the reinforcement and sharp corners and edges. Rodding is done continuously over the complete area to effectively pack the concrete and drive away entrapped air. Sometimes, instead of iron rod, bamboos or cane is also used for rodding purpose.

Ramming should be done with care. Light ramming can be permitted in unreinforced foundation concrete or is permitted in case of reinforced concrete or in the upper floor construction, where concrete is placed in formwork supported on

struts. If ramming is adopted in the above case the position of reinforcement may be disturbed or the formwork may fail, particularly, if steel rammer is used.
Tamping is one of the usual methods adopted in compacting roof or floor slab or road pavements where the thickness of concrete is comparatively less and consists of beating the top surface by wooden cross beam of section about 10 cm x 10 cm. Since the tamping bar is sufficiently long it does not only compact, but also levels the top surface across the entire length.

1.3.3 Compaction by vibration:

A review of the literature on the effect of vibration on concrete quality revealed that vibration of fresh concrete during casting is critical. Vibration has been shown to be the most effective method for bringing concrete particles together into a compact mass. However, it is not uncommon for concreting to take place near a source of vibration that may last for the duration of the setting time and early age of the concrete.

A number of studies on the effect of re-vibration have recently been published, indicating that it may produce benefits, particularly for wetter mixtures, in eliminating water gained under reinforcing bars and reducing bug holes, all of which will increase the strength.

The purpose of this experimental study was to investigate the effects of vibration, delaying the re-vibration, and repeated vibration of concrete in order to gain a better understanding of the effects and processes involved. The influence of Kurdistan's hot, dry climate and the time of useful vibration are particularly considered.

Research by Allen Hulshize indicates that vibration on fresh and maturing concrete does not affect its properties. However, his investigation maintained such a broad focus on the vibration of concrete that specific time periods may need further investigation, Bastion, considered the effects of vibratory concrete during the setting period (Allen 1970).

Due to decreased floor mass and longer span lengths, floor vibrations have become an area of concern. Design criteria (Allen and Rainer, Allen: 1990 b, Murray, 1991) are available to help designs minimize annoying vibrations in floor systems. In general, hours that comply with the criteria and are used for their original purpose are found to be acceptable to the occupants.

It is pointed out that the compaction, if properly carried out on concrete with sufficient workability, gives satisfactory results, but the strength of the hand compacted will be necessarily low because of higher water cement ratio required for full compaction. Where high strength is required, it is necessary that stiff concrete, with low water/cement ratio be used. To compact such concrete, mechanically operated vibratory equipment, must be used. The vibrated concrete with low water/cement ratio will have many advantages over the hand compacted concrete with higher water/cement ratio.

The modern high frequency vibration makes it possible to place economically concrete which is impracticable to place by hand. A concrete with about 4 cm slump can be placed and compacted fully in a closely spaced reinforced concrete work, whereas, with hand compaction, much higher consistency say about 12 cm slump may be required. The action of vibration is to set the particles of fresh concrete in motion, reducing the friction between them and affecting a temporary liquefaction of concrete which enables easy settlement. While vibration itself does not affect the strength of concrete which is controlled by the water/cement ratio, it permits the use of less water.

Compaction of concrete by vibration has almost completely revolutionized the concept of concrete technology, making possible the use of low slump stiff mixes for production of high quality concrete with required strength and impermeability. The use of vibration may be essential for the production of good concrete where the congestion of reinforcement or the inaccessibility of concrete in the formwork is such that hand compaction methods are not practicable. Vibration may also be necessary if the available concrete is of such poor workability unless large amount of water and cement is used. In this way, vibration under suitable conditions, produce better quality concrete than by hand compaction. Low cement content and lower water cement ratio can produce equally strong concrete more than by hand compaction.

Although vibration properly is a great step forward in the production of quality concrete, it is more often employed as a method of placing ordinary concrete easily than as a method of obtaining high grade concrete at an economical cost. All potential advantages of vibration can be fully realized only if proper control is exercised in the design and manufacture of concrete and certain rules are observed regarding the proper use of different types of vibration.

1.3.3.1 Internal vibration:

Internal concrete vibrators, also known as immersion vibrators, are widely used for consolidation all over the world. They consist of a vibrating poker as shown in figure 1-1 with diameter varying between 20 mm and 150 mm (CCANZ, 2005). The poker on the internal vibrator is inserted vertically into the cast concrete at pre-determined spacing.

Fig. 1-1 Showing a flexible shaft poker vibrator

The spacing and depth is usually determined based on the radius of the poker used for the vibration. The sleek nature of these vibrators makes it efficient and easy to access all the necessary areas during consolidation. Different types of internal vibrators can be used such as flexible shaft, pneumatic and hydraulic vibrators.

The most commonly used internal vibrator is the flexible shaft vibrator. This type of vibrator is usually driven by a pneumatic motor that runs using combustible fuel or electricity (ACI 301- 05, 2005). The frequency of the internal vibrator must not be less than 120 Hz during any phase in the compaction of concrete (SANS 5862-4, 2006). Table 1-1 shows the different groups of internal vibrators available in the market

Table 1-1 Application and characteristics of internal vibrators

			Suggested value	Approximate value	
Group	Diameter in mm	Recommended frequency (vpm)	Average amplitude in mm	Radius of action in mm	Rate of concrete placement (Yards/hr)
1	20-40	9000-15000 (150-250 Hz)	0.4-0.8	150-610	5-15
2	30-60	8500-12500 (141-208 Hz)	0.5-1.0	510-810	12-45
3	50-90	8000-12000 (133-200 Hz)	0.6-1.3	710-1220	24-60

1.3.3.2 External vibrator (Formwork vibrator):

The basis for this type of vibration involves vibration units with an electric motor and variable power outputs. These provide the centrifugal force and wattage required for the process (CCANZ, 2005). This type of vibration is used commonly during the casting of pre-cast concrete and concrete with high reinforcement concentration.

Unlike an internal concrete vibrator, the external vibrator cannot be moved easily as it has to be mounted. The external vibrators are directly mounted to the form wall without touching the concrete (Concrete Vibration Handbook, 2003). Paiovici (2004) states that the form required for external vibration should be water tight in order to prevent loss of water and achieve smooth finishing.

External vibrators as shown in figure 1-2a usually operate at a frequency between 50 Hz to 200Hz. To obtain the best results during consolidation, high frequency must be used with low vibration amplitude (ACI 301-5, 2005). The effectiveness of the external vibrator also depends on how efficiently the acceleration and the centrifugal force are conveyed from the vibrator to the form. Smaller accelerations in the order of 1g to 2g are required for workable mixes while stiffer mixes require accelerations between 3g to 5g (ACI 301-5, 2005). In addition to acceleration, proper distance must be maintained between the external vibrators for a more uniform consolidation. The nominal distance between vibrators can be in the range of 1.5m to 2.5m depending on the interference between the vibrators radii of action (ACI 301-5, 2005). However, internal vibration must be provided for some regions lacking adequate consolidation usually around the areas of over lapping radii of action.

1.3.3.3 Table vibrator:

This type of vibration involves a platform that freely resonates as shown in figure 1-2b. During vibration, both the form and concrete move freely according to the amplitude of the pressure waves (CCANZ, 2005). The vibrations from the table hit the surface of the settling concrete, eliminating the voids. The table size varies according to the required specification. The frequencies of the vibration table vary between 30 Hz to 150 Hz (ACI 301-5, 2005). However, according to SANS 5862-4 (2006), the frequency of the table vibrator must not be less than 50 Hz during consolidation of concrete.

Table vibrators are commonly used in laboratories for concrete casting as they allow both the form and concrete to vibrate simultaneously. The vibration is transmitted from the table to the mold and then to the concrete in the mold. As the platform of the table vibrates, air bubbles begin to appear on the surface of concrete. Proper consolidation is achieved when the air bubbles seize appearing. The process usually takes between 10 seconds to 30 seconds of vibration for proper consolidation with minimum or no voids.

(a) (b)

Fig. 1-2 Showing an external vibrator (a) and a vibrating table (b)

1.3.3.4 Platform vibrator:

Platform vibrator larger in size. This is used in the manufacture of large prefabricated concrete elements such as electric poles, railway sleepers, prefabricated roofing elements etc. Sometime, the platform vibrator is also coupled with jerking or shock giving arrangements such that a thorough compaction is to the concrete.

1.3.3.5 Surface Vibrators:

Surface vibrators are sometimes known as, (Screed Board Vibrator). A small vibrator placed on the screed board gives an effective method of compaction and leveling of thin concrete members, such as floor slabs, roof slabs and road surface. Mostly, floor slabs and roof are so thin that internal vibrator or any other type of vibrator cannot be easily employed. In such cases, the surface vibrator can be effectively used. In general, surface vibrators are not effective beyond about 15cm. Sometimes, the concrete is vibrated by using vibratory roller moved on the surface. Vibrating roller is used for compaction of road slabs.

1.4 Procedure for Re-vibration

The technique behind using and operating concrete vibrators is very important in achieving optimum results. Internal vibration is chosen to execute the re-vibration of concrete. Based on my personal site experience during vacation training, it was noticed that several concrete work men did not follow the standard vibration procedures. The common faults noticed were as follows: applying the poker at an angle as opposed to perpendicular to the surface of concrete, poking the concrete randomly rather than at regular intervals and stopping the vibration before the air bubbles completely seize to appear. This can affect the quality of concrete especially the compressive strength. Poor vibration of concrete can also lead to cracks, voids and poor aggregate bonding. According to SANS 5862-4 (2006), the recommended frequency for internal vibration must not be less than 120 Hz. Vibration is used as a method of compaction for concrete with a slump ranging between 0 mm to 75 mm (SANS 5861-3, 2006). However, if the slump exceeds 75 mm, compaction by tamping is preferred (SANS 5861-3, 2006).

Vibration and re-vibration are carried out in a similar manner expect that vibration is done while casting and re-vibration is done after a time lag from casting. The technique to achieve the best possible results while vibrating is to observe the vibration action of the poker and adjust the radius of action of the concrete vibrator accordingly, creating an overlap enough to cover the entire surface of concrete

(Concrete Vibration Handbook, 2003). The vibrator must be kept long enough till the air bubbles are no longer released to the surface (SANS 5861-3, 2006) as shown in figure 1-3.

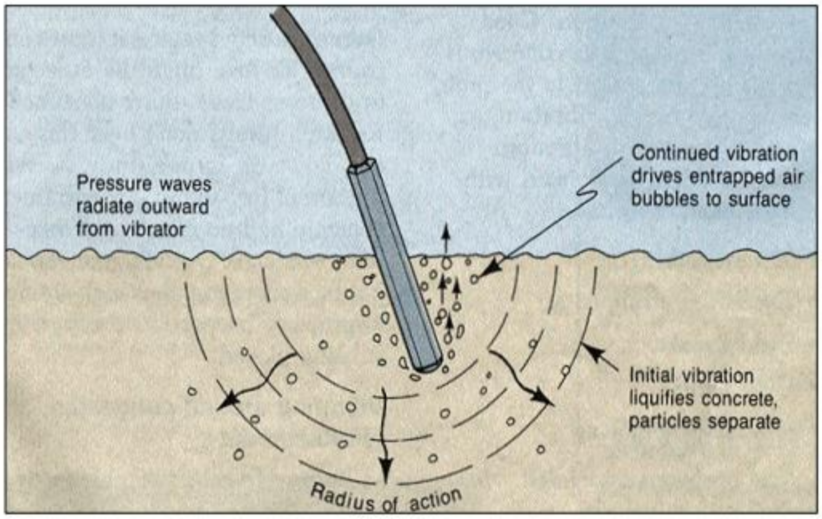

Fig. 1-3 Vibrator in action during casting

The internal vibrator must be able to penetrate the surface of concrete under its self-weight while re-vibrating. This is to ensure that the concrete has not fully set. The head of the vibrator must be completely submerged while monitoring the spacing and time span of the vibrations (Concrete Vibration Handbook, 2003). After vibration the equipment must be pulled out gently in order to avoid cold joints. This process is repeated till all the air bubbles disappear.

1.5 General Points on Using Vibrators:

Vibrators may be powered by any of the following units:

(a) Electric motors either driving the vibrator through flexible shaft or situated in the head of the vibrator.

(b) Internal combustion engine driving the vibrator needle through flexible shaft.

(c) Compressed-air motor near the head of the vibrator.

Where reliable supplies of electricity are available the electric motor is generally the most satisfactory and economical power unit. The speed is relatively constant, and the cables supplying current are light and easily handled.

Small portable petrol engines are sometimes used for vibrating concrete. They are more easily put out of action by site conditions. They are not so reliable as the electric or compressed-air motors. They should be located conveniently near the works to be vibrated and should be properly secured to their base.

Compressed-air motors are generally quite suitable but pneumatic vibrators are sometimes difficult to manipulate when the compressor cannot be placed adjacent to the work such as on high scaffolding or depths below ground level due to the heavy weight of air hoses.

Compressed-air vibrators give trouble especially in cold weather, by freezing at exhaust unless alcohol is trickled into the air or dry air is used. Glycol type anti-freeze agents tend to cause gumming of the vibrator. There is also a tendency for moisture to collect in the motor, hence care should be taken to remove the possible damage.

The speed of both the petrol and compressed-air motors tend to vary giving rise to variation in the compacting effect of the vibrator.

1.6 Further Instructions on the Use of Vibrators:

Care shall be taken that the vibrating head does not come into contact with hard objects like hardened concrete, steel and wood, as otherwise the impact may damage the bearings. The prime mover should be, as far as possible, started only when the head is raised or resting on soft support. Similar precautions shall be observed while introducing or withdrawing the vibrator in the concrete to be consolidated. When the space for introduction is narrow, the vibrator should be switched on only after the vibrator head has been introduced into the concrete. Unnecessary sharp bends in the flexible shaft drive shall be avoided.

Vibrators conforming to the requirements of ISS 2505-1963 (i.e specification for concrete vibrators, immersion type) shall be used. The size and characteristics of the vibrator suitable for a particular job vary with the concrete mix design, quality and workability of concrete, placing conditions, size and shape of the member and shall be selected depending upon various requirements.

Correct design of concrete mix and an effective control in the manufacture of concrete, right from the selection of constituent materials through its correct proportioning to its placing, are essential to obtain maximum benefits of vibration. For best results, the concrete to be vibrated shall be of the stiffest possible consistency, generally within a range of 0.75 to 0.85 compacting factor, provided the fine mortar in concrete shows at least a greasy wet appearance when the vibrator is slowly withdrawn from the concrete and the material closes over the space occupied by the vibrator needle leaving no pronounced hole. The vibration of concrete of very high workability will not increase its strength, it may on the contrary, cause segregation.

For vibrated concrete, the formwork shall be stronger than is necessary for hand compacted concrete and greater care is exercised in its assembly. It must be designed to take up increased pressure of concrete variations caused in the neighborhood of the vibrating head which may result in excessive local stress on the formwork. More exact details on the possible pressures are not available and much depend upon experience, judgment and the character of works. The joints of the formwork shall be made and maintained tight and close enough to prevent the squeezing out of grout or sucking in of air during vibration. Absence of this precautions may cause honey-combing in the surface of concrete, impairing the appearance and sometimes weakening the structure.

The amount of mortar leakage or the permissible gap between sheathing boards will depend on the desired final appearance of the works but normally gaps more than 1.5 mm between the boards should not be permitted. Sometimes even narrower joints may be objectionable from the point of view of their effect on the surface be made as small as possible by making the shutter sections large. Applications on the formwork, if any, to prevent the adhesion on concrete should be very thin as otherwise they may mix with the concrete under the effect of vibration. The vibrator may be vertically, horizontally or at an angle depending upon the nature of the job. The concrete to be vibrated shall be placed in position in level layers of suitable thickness not greater than the effective length of the vibrator needle.

The concrete at the surface must be distributed as horizontally as possible, since the concrete flows in slopes while being vibrated and may segregate. The internal vibrator should not be used to spread the concrete from the filling as this can cause considerable segregation of concrete.

It is advisable to deposit concrete well in advance of the point of vibration. This prevents the concrete from subsiding non-uniformly and prevents the formation of incipient plastic cracks.

1.6.1 Height of Concrete Layer:

Concrete is placed in thin layers consistent with the method being used to place and vibrate the concrete. Usually concrete shall be placed in a thickness not more than 60 cm and on initial placing in thickness not more than 15 cm. The superimposed load increasing with height of the layer will favor the action of the vibrator, but as it is also the path of air forced upwards, it may trap air rising up by vibration. Very deep layers (Say more than 60 cm) should be avoided although the height of layer can also be one meter provided the vibrator used is sufficiently powerful.

1.6.2 Speed of Insertion and Withdrawal of the Vibrating Head:

The vibrating head shall be regularly and uniformly inserted in the concrete so that it penetrates of its own accord and shall be withdrawn quite slowly whilst still running so as to allow redistribution of concrete in its water and allow the concrete to flow faster into the hole behind the vibrator. The rate of withdrawal is determined by the rate at which the compaction in the active zone is completed. Usually a speed of 3 cm/s gives sufficient consolidation without undue strain on the operator.

1.6.3 Duration of Vibration:

New filling shall be vibrated while the concrete is plastic, preferably within one hour. The duration of vibration in each position of insertion is dependent upon the height of the layer, the size and characteristics of the vibrator and workability of the concrete mix. It is better to insert the vibrating head at a number of places than to leave it for a long time in one place, as in the latter case, there is a tendency for formation of motor poker at the point of insertion of the vibrator.

The vibrator head shall be kept in one position till the concrete within its influence is completely consolidated which will be indicated by formation of circular shaped cement grout on the surface of concrete, appearance of flattened glistening surface and cessation of the rise of entrapped air. Vibration shall be continued until the coarse aggregate particles have tended into the surface but have not disappeared.
The time required to an effect complete consolidation is readily judged by the experienced vibrator operator through the feel of the vibrator, resumption of frequency of vibration after the short period of dropping off frequency when the vibrator is first inserted.

1.6.4 Over-vibration:

There is a possibility of over-vibration while trying to achieve thorough vibration, but it is exceedingly unlikely in well-proportioned mixes containing normal weight aggregates. Generally, with proper mixes, extended vibration will be only a waste of effort without any particular harm to the concrete.

However, where the concrete is too workable for the conditions of placing, or where the quantity of mortar is un excess of the volume of voids in the coarse aggregate, or where the grading of aggregate is unsatisfactory, over-vibration will encourage segregation, causing migration of the lighter and smaller constituents of the mix to the surface, thereby producing layer of mortar or laitance on the surface and leakage of motor through the defective joints in the formwork. This may produce concrete with poor resistance to abrasion and attack by various agencies, such as frost, or may result in planes of weakness where successive lifts are being placed. If over vibration occurs, it will be immediately evident to an experienced vibrator operator or supervisor by a frothy appearance due to the accumulation of many small air bubbles on the surface. These results are more liable to occur when the concrete is too wet and the proper correction will be to reduce the workability (not the vibration), until the evidence of over-vibration disappears during the amount of vibration judged necessary to consolidate the concrete and to eliminate air-bubble blemishes.

1.7 Re-vibration:

Re-vibration is delayed vibration of concrete that has already been placed and compacted. It may occur while placing successive layers of concrete, when vibrations in the upper layer of fresh concrete partially hardened or may be done intentionally to achieve certain advantages.

Except in the case of exposed concrete and provided the concrete becomes plastic under vibration, re-vibration is not harmful and may be beneficial. By repeated vibration over a long period (repetition of vibration earliest after one hour from the time of initial vibration), the quality of concrete can be improved because it rearranges the aggregate particles and eliminates entrapped water from under the aggregate and reinforcing steel, with the consequence of full contact between mortar and coarse aggregate or between steel and mortar and thus produces stronger and watertight concrete. Plastic shrinkage cracks as well as other disturbances like hollow space below the reinforcement bars and below the coarse aggregate, can thereby be closed again provided the concrete becomes soft again when the vibrator head is introduced. Re-vibration of concrete results in improved compressive and bond strength, reduction of honey-comb, release of water trapped under horizontal reinforcing bars and removal of air and water pockets.

Re-vibration is most effective at the lapse of maximum time after the initial vibration, provided the concrete is sufficiently plastic to allow the vibrator to sink under its own weight into the concrete and make it momentarily plastic.

1.8 Mechanism of Re-Vibration of Concrete

The process of intentionally and systematically vibrating the placed concrete again after its consolidation is complete is known as re-vibration of concrete. A properly executed re-vibration results in improved concrete quality, such as increased strengths and bonds, better impermeability, less shrinkage and creep, less surface and other voids, and fewer cracks in fresh concrete, among other benefits. Re-vibration can usually be done at any time as long as the running internal vibrator can sink by its own weight into the concrete or the external vibrator or vibrating table can temporarily liquefy the concrete. When the penetration resistance of the standard steel needle specified in ASTM C 403, reaches 3.5 N/mm2, the stiffness limit for re-vibration is usually accepted. Re-vibrating the concrete temporarily liquefies it. The formation of calcium hydroxide $Ca(OH)_2$, which typically accounts for 15% to 25% of Ordinary Portland Cement concrete, is the primary chemical process that occurs within the first two hours after the concrete is placed. The other major hydration product is calcium silicate hydrate (***C-S-H***), which makes up about 50% of OPC concrete and gives it hardness and durability. When re-vibration occurs after the initial set, it breaks down some of the calcium hydroxide $Ca(OH)_2$ that has already formed, allowing freshly placed concrete adjacent to the re-vibrated concrete to join with it rather than introducing a construction joint and allowing the structure to become a monolithic concrete structure once more.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter discusses the relevant information relating to the effect of re-vibrating concrete on the mechanical properties. The key theories and concepts encountered during the desk study are discussed. The relevant experimental data from prior researchers is used as a basis for comparison.

2.2 History of consolidating concrete

Engineers often link the increase or decrease in compressive strength to the water-cement ratio used in the mix design. Concrete tests ran by (Suprenant, 1988) showed the effect of varying water-cement ratios on the compressive strength. The experiments were performed on concrete by varying the duration of compaction or vibration.

In the late 1930's, it was believed that the re-vibration of concrete would disturb the hardening process affecting the quality of the construction (Tuthill, 1977). However, some engineers continued to disturb the setting concrete due to delays during massive concrete pouring. The concrete mixture would be tamped and agitated till the arrival of fresh concrete (Bhaskar, Kumar and Rao, 2008). The reasoning behind the application of this process was that as the vibrator penetrates the surface of the concrete layer under its own weight, it causes the set layer of calcium hydroxide to breakdown allowing the initial surface layer to merge easily with the freshly cast concrete.

(Tuthill and Davis, 1938) they reported in this study that as long as the concrete remains in a fairly plastic state, say up to two hours after mixing, slight inadvertent re-vibration, would cause no damage and may even benefit. A small amount of test data was presented to back up this claim. Concrete in 6 x 12 inch cylinders was successfully re-vibrated after intervals as long as 10 hours from the time of mixing, resulting in compressive strength increases of up to 25%, with the greatest strength increases obtained when the interval between time of mixing and time of re-vibration was between 2 and 5 hours. They also reported bond data that showed that re-vibration 2 to 5 hours after casting could result in a 30% to 50% increase in bond strength for plain bars and a maximum of 100% increase in bond strength for deformed bars. (Purandare, N. N., 1946) was conducting similar studies at the University of London at the same time this work was reported, though his findings were not published until 1946. Both cubes and beam specimens were cast in l: 2: 4 concrete (water cement ratio of 0.55) and re-vibrated at 1 to 5 hour intervals. Specimens were tested at 7 days of age. He reported a maximum compressive strength increase of 34.3 percent and a maximum ultimate beam strength increase of 35.2 percent when re-vibration occurred two hours after casting.

In stark contrast to the preceding, (Larnach, 1952) reported that his results revealed no tendency for re-vibration of partially set concrete to improve either compressive or bond strength. The time between casting and re-vibration in his studies ranged from 15 minutes to 6 hours. One possible explanation for his negative results is that he used very dry mixes, so re-vibration may not have brought them back to a plastic state.

In 1977, Tuthill demonstrated that re-vibration of concrete is beneficial as it increases the compressive strength of concrete. Two mixes are shown in figure 2-1, one containing voids (air-entrained) and the other without voids (non-air-entrained). The experiment was performed using the 6 bag ordinary Portland cement mix at room temperature (20 ^{o}C-23 ^{o}C) while keeping the water-cement ratio constant. The results obtained showed an increase in 7 day compressive strength by 36% after a time lag of 4 hours (Tuthill, 1977).

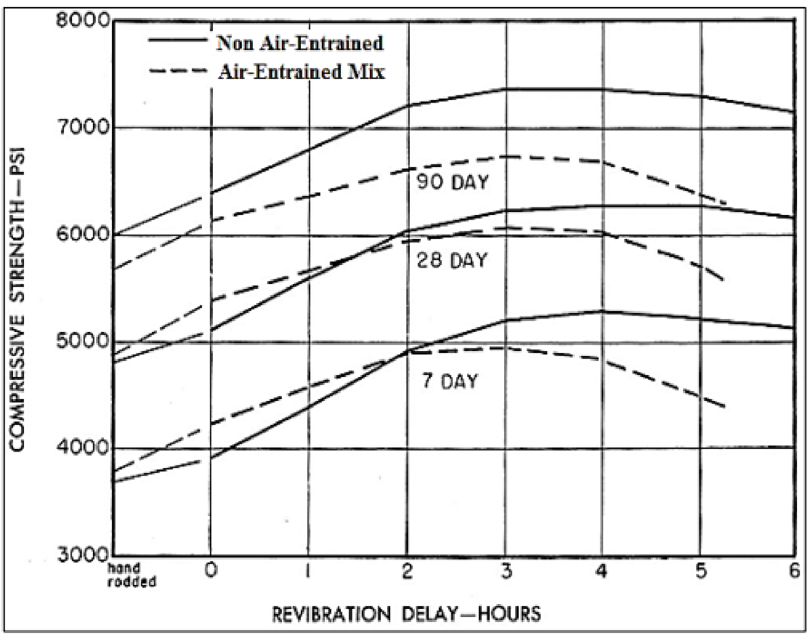

Fig. 2-1 Relation between compressive strength and re-vibration delay

(Vollick, 1958) extended the study of re-vibration in 1958 to mixes containing admixtures causing air entrainment, retardation of set, or both air entrainment and retardation of set. All mixes were proportioned for a three-inch slump, and vibration was induced using a 6000-cycle-per-minute spud vibrator. The specimens of each mix were first vibrated and then re-vibrated at one, two, three, and four hour intervals. An impact hammer was used to test the strength of one series of specimens, and the average readings were used for the indicated strengths. Figure 2-2 depicts the results of this test.

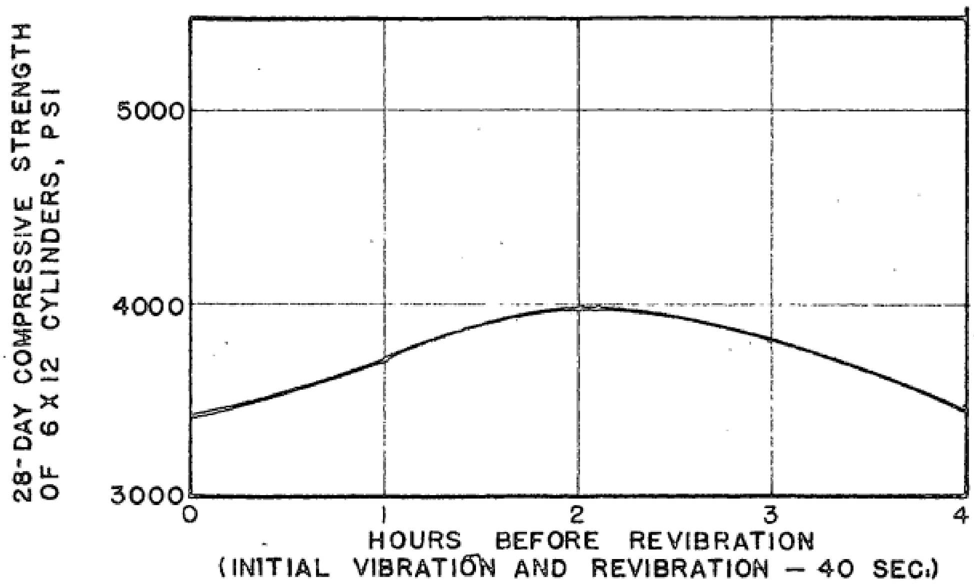

Fig. 2-2 Effect of re-vibration on strength of plain concrete

2.3 Re-vibration mechanism

(Bhaskar, Kumar and Rao, 2008) later showed similar results using controlled experiments while varying the water-cement ratios at room temperature. However, (Bhaskar, Kumar and Rao, 2008) used ordinary Portland cement with different properties such as initial and final setting times than that used by (Tuthill, 1977). Re-vibration also showed additional benefits such as decreased permeability, reduction in voids, reduction in shrinkage and decreased surface cracks (Bhaskar, Kumar and Rao, 2008).

After a specific time, the compressive strength reaches its optimum value and there is no further positive influence of re-vibration on the concrete. According to the results shown in figure 2-3 compressive strength decreases below the control value after the final setting time of concrete. The reason for the decrease is that after the final setting time which is usually between 2 to 3 hours, calcium silicate hydrates in concrete form a strong chemical bond with the aggregates. Once the bonds are formed, re-vibration disrupts the bond structure weakening the compressive strength of concrete.

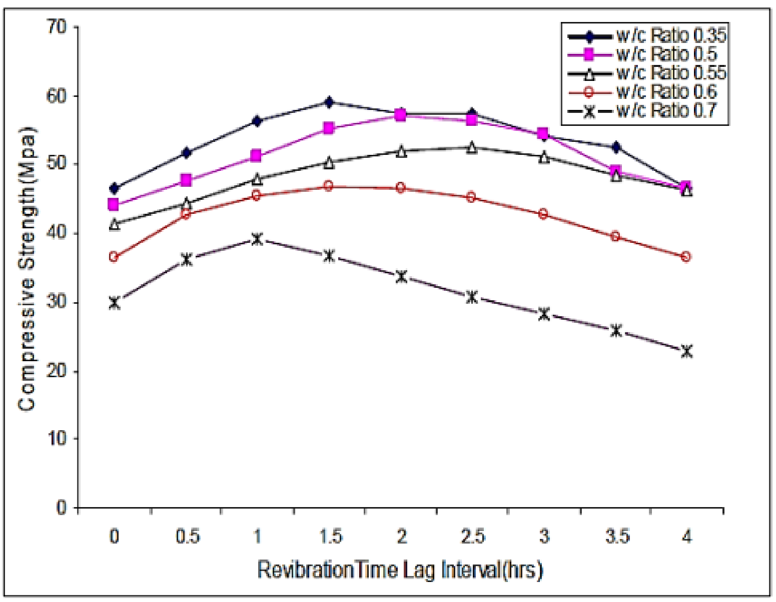

Fig. 2-3 Relation between compressive strength, w/c ratio and re-vibration lag

(Bhaskar, Kumar and Rao, 2008) also showed that the percentage increase in compressive strength was most effective for the concrete mixture with a high water-cement ratio of 0.7 at 31.4%. However, the maximum percentage increase in the mixtures with water-cement ratio of 0.35 was 27%, 0.5 was 29.1%, 0.55 was

27.3% and 0.6 was 28.5%. This data is shown in figure 2-4. As the re-vibration time increases, the percentage increase in compressive strength decreases. The most affected mix as the re-vibration time lag increases was noted to be the one with a high water-cement ratio of 0.7. Re-vibration of the high water-cement ratio mix caused a decrease in compressive strength by 25% after a 4 hour's time lag.

The increase in compressive strength is due to the vibrator breaking the calcium hydroxide layer before the initial setting time of concrete making the bond formation stronger after re-vibration. The results, as stated above, showed no solid relationship between the water cement ratio and the percentage increase in compressive strength. (Bhaskar, Kumar and Rao, 2008) prove the benefits of re-vibration to concrete compressive strength. However, other factors such as changes in temperature impact the ability of re-vibration and need to be examined as temperatures keep fluctuating on construction sites during concrete pouring.

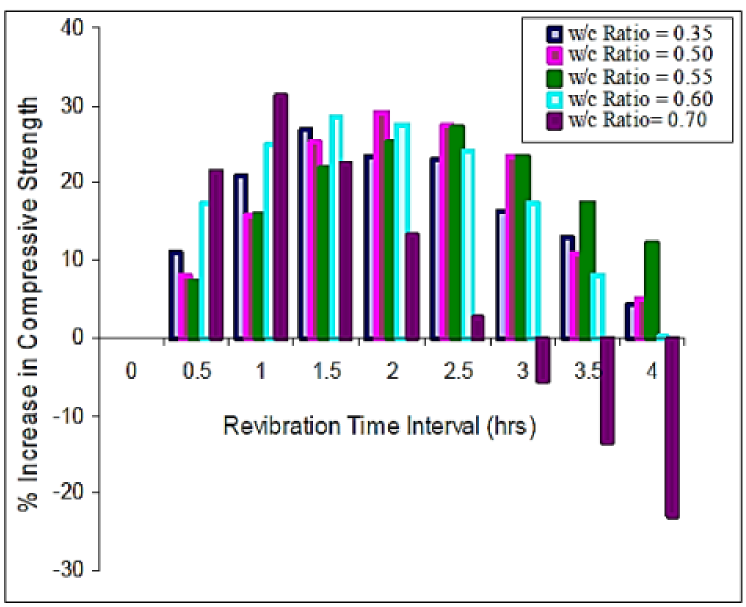

Fig. 2-4 % increase in compressive strength versus re-vibration lag

(Kummetha and beushausen, 2014) A series of laboratory experiments were designed to determine the effects of re-vibration of concrete on its compressive strength when varying the cement type, water-cement ratios and mix temperature. 240 concrete cubes (150 mm x 150 mm x 150 mm) were cast, cured and tested for compressive strength at varying water-cement ratios (0.4, 0.6) and mix temperature (10°C, 20°C and 30°C). Cores were then taken from some of the cubes to identify any damage caused by re-vibration to the internal structure of the concrete. The compressive strength of the cubes is analyzed using graphs based on the cement type used in the mix. Re-vibration time lag is plotted against the compressive strength for the different concrete mix temperatures. The percentage increase in compressive strength after re-vibration of concrete is tabulated in table 2-1 and shown in figure 2-5.

Table 2-1 % increase in compressive strength of CEM I concrete

Temperature		10° C	20° C	30° C
w/c	Re-vibration time/ hr	% increase in strength	% increase in strength	% increase in strength
0.4	1	16.7	24.9	24.0
	2	31.9	29.5	20.2
	4	28.3	22.7	10.1
	6	27.5	22.2	6.0
0.6	1	10.0	13.0	12.0
	2	15.2	15.4	7.1
	4	0.4	1.0	-1.2
	6	-3.3	-9.6	-6.2

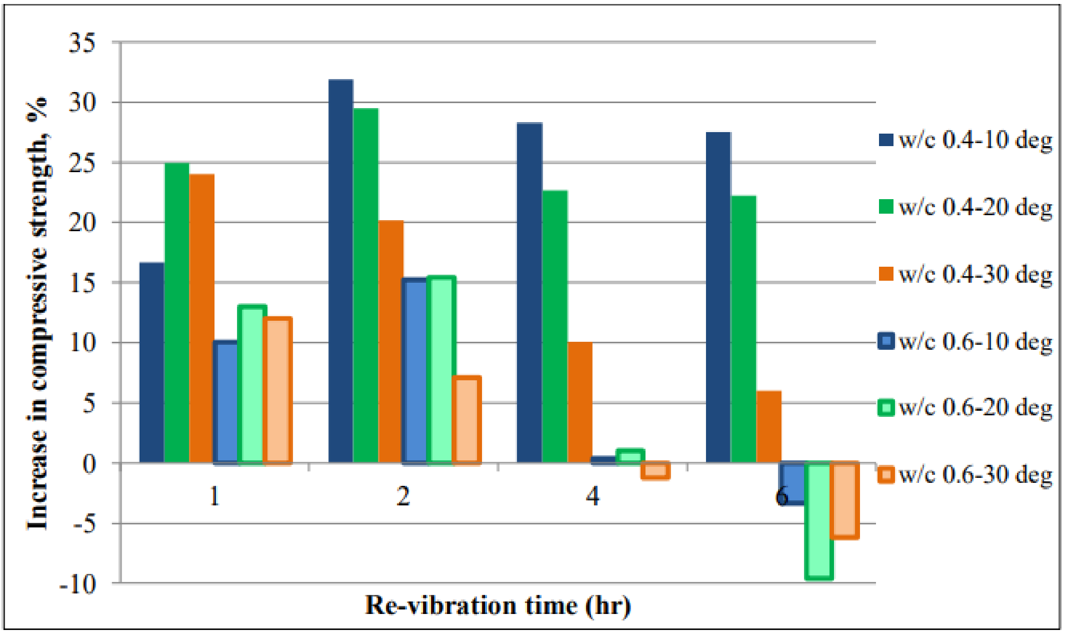

Fig. 2-5 Percentage increase in compressive strength

As seen in figure (2-5), increase in percentage compressive strength decreases as the re-vibration time increases.

The obtained results after the experimentation showed that the compressive strength of the concrete increased up to when the peak re-vibration time was reached. After which it began to decrease depending on the mix temperature used.

2.4 Influence of re-vibration on chemical structure of concrete

The main constituents of ordinary portland cement are calcium silicates (C3S and C2S). As the cement reacts with water, it forms calcium silicate hydrates and calcium hydroxide. In ordinary Portland cement, calcium hydroxide is formed initially in the first two hours of concrete casting (Bhaskar, Kumar and Rao, 2008).This is then followed by the formation of calcium silicate hydrates at a later stage. A simplified example of the hydration of Tri-calcium silicate is demonstrated below CSIR, 2010):

$$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3CH$$

The process of re-vibration breaks down the layer of calcium hydroxide (CH) formed on the surface due to the lag between concrete pouring and allows for the new mixture to bond with the previous layer (Bhaskar, Kumar and Rao, 2008). This prevents the formation of a joint between the two layers of concrete, making the structure combined and uniform.

The compositions of cement vary based on the blend, type and strength. The quantity of calcium hydroxide and calcium silicate hydrates also varies depending on the quantity of calcium oxide present in the cement. The strength of concrete is mainly achieved because of the calcium silicate hydrates formed.

2.5 Mechanisms by which Re-vibration increases strength

At the moment, several theories have been proposed as to the precise mechanism by which re-vibration increases the strength of concrete. The following paragraphs will go over each of these.

A- (Sawyer and Lee, 1956) proposed one of the most popular theories, namely that re-vibration causes the mortar and concrete to become more densely consolidated, allowing for more advantageous deployment of hydration products. This is certainly consistent with the general theory that strength is a function of void cement ratio, with any decrease in this ratio resulting in an increase in strength. This also corresponds to (Tuthill and Davis, 1938) observations that when the concrete is vibrated at an early stage (before the initial set of cement), a noticeable amount of air and water may be expelled from the mass, resulting in better consolidation.

B- Another theory put forward by (Sawyer and Lee, 1956) is that "the vibrating disturbances in some way accelerate and extend the production and consequently increase the amount of strengthening hydrates at the particular ages up to 90 days". "when the hydration of cement starts the initial structure is determined by either C3A or C3S which ever hydrates first. In a normally retarded set, the C3S is presumed to hydrate first and to establish the primary structural bonds. If, on the other hand C3A hydrates first, re-vibration may displace this weaker structure and allow the normal structure of C3S to develop otherwise normally retarded set. The extent of hydration of C3S is relatively insignificant up to 4 to 6 hours while that of C3A may be (unless normally retarded) almost complete within 10 hours. These factors could have a significant influence on the rate of strength development and even the ultimate strength". According to this theory, re-vibration tends to break the gel structure surrounding hydrating cement particles, allowing more ready access of water to the un-hydrated portions of the cement particles and allowing for more hydration.

C- (Neville, 1963) proposes that the increase in strength caused by re-vibration may be due to a reduction in plastic shrinkage stresses around aggregate particles caused by re-vibration. If this theory is correct, re-vibration should not result in any strength increases (or at least only minor strength increases) in plain cement pastes.

D- (Lea et al., 1956) proposes that re-vibration increases strength by facilitating the expansion and dispersion of the cement paste. He describes the mechanism as follows: "After hardening has begun, the rigid mass can no longer accommodate a localized growth of the solids around the cement grains, and an expansion occurs if a supply of water is kept available to continue hydration. And, because the saturated cement gel expands (about 2.2 times) during hydration, re-vibration of concrete after this expansion has begun will or should facilitate the expansion and dispersion of the cement paste. As previously stated, dispersion should increase strength."

E- With the formation of gel, calcium hydroxide crystals appear immediately after the addition of water to the cement (silicate hydrates). When the paste is re-vibrated before it hardens, these crystals break, resulting in an uneven surface on each cement particle. As a result of the vibrations, the cement particles get a better grip on each other, resulting in better consolidation and an increase in strength.
F- Another theory is that re-vibration reduces the pressure due to crystals at the outer surface while increasing strength due to better consolidation.

G- The effect of re-vibration on bleeding was also considered as a factor in the increased strength. Some of the water comes to the surface of the concrete (or paste or mortar) due to re-vibration, resulting in a lower water-cement ratio in the mass and thus an increase in strength.

CHAPTER THREE

EXPERIMENTAL PROGRAME

3.1 Introduction

This chapter describes in detail the experimental program used to evaluate the impact of applying re-vibration to concrete beams. The experimental method followed the procedure for evaluating the re-vibration process of reinforced concrete beams. The chapter describes the experimental design, the testing equipment, and the properties of the materials used. Several tests were also performed to confirm the effect of the re-vibration technique on the mechanical properties of concrete.

The experimental program's results were integrated into a database that was used to determine the factors that influence the development of reinforced beams after the casted concrete was re-vibrated. These results were compared to those of unreinforced mixes that did not receive a second vibration. The loading process for all the beams were videoed for the purpose recording crack behavior of the beams. This chapter also contains the information's about the properties of the materials, preparation of samples, mixing and casting the reinforced concrete beam, and testing procedure.

3.2 Material Properties

3.2.1 Cement

Fresh concrete is a mixture of water, cement, and aggregate. It is important that the constituent materials remain uniformly distributed within the concrete mass during the various stages of its handling and that full compaction is achieved.

When either of these conditions is not satisfied the properties of the resulting hardened concrete, for example, strength and durability, are adversely affected.

The characteristics of fresh concrete which effect full compaction are its consistency, mobility and compatibility. In concrete practice these are often collectively known as workability. The ability of concrete to maintain its uniformity is governed by its stability, which depends on its consistency and its cohesiveness. Since the methods employed for conveying, placing and consolidating a concrete mix, as well as the nature of the section to be cast, may vary from job to job it follows that the corresponding workability and stability requirements will also vary. The assessment of the suitability of fresh concrete for a particular job will always to some extent remain a matter of personal judgment.

Concrete used in this research was made using a mixture of only three materials: aggregate, Portland cement, and potable water. The ordinary Portland cement from Mass company has been employed to design the normal concrete mixes. The physical and chemical properties of the cement are tested and checked according to the specification of ASTM (ASTM- C150). As shown in the Table (3-1 and 3-2)

Table 3-1 Physical Properties of the cement

Physical Tests	Results	ASTM C150-10
Initial setting time	180 min.	At least to be 45min.
Final setting time	245 min.	Not more than 600min.
Compressive strength 3 days age	22.68	14.7 MPa, a lower limit

Compressive strength 28 days age	32.25	22.5 MPa, a lower limit
Specific gravity	3.15	
Density	1400 kg/m^3	

Table 3-2 Chemical tests for cement investigated by (Directorate of Erbil Construction Laboratory)

Chemical tests	Results	Specification
Lost in ignition	2.22%	4% Max.
Insoluble material	0.5%	1.5 Max.
Si$O2$	20%	
Cao	63.5%	
$AL2O3$	4%	
$Fe2O2$	4.5%	
MgO	2.15%	5% Max.
S$O3$	2.1%	28% Max.
$C3$A	3%	
L.S.F	0.97%	(0.66 – 1.02)
$C3$S	67.2%	
$C2$S	6.9%	
$C4$AF	13.7%	

3.2.1.1 Concrete Mixing:

The materials must be thoroughly mixed in order to produce uniform concrete. The mixing should make the mass homogeneous, uniform in color, and consistent. Concrete can be mixed in two ways:

(a) Hand mixing.

Hand mixing is used for small-scale, unimportant concrete projects. Because the mixing cannot be thorough or efficient, it is preferable to add 10% more cement to compensate for the inferior concrete produced by this method.

(b) Machine mixing.

Concrete mixing is almost always done by machine for reinforced concrete work and medium or large scale mass concrete work. When the amount of concrete to be produced is large, machine mixing is not only efficient but also cost effective. In this study we took a machine mixer for our test.

3.2.1.2 Mixing time:

Concrete mixers are typically designed to spin at 15 to 20 revolutions per minute. In a well-designed mixer, approximately 25 to 30 revolutions are required for proper mixing. The normal tendency on the job site is to shorten the mixing time in order to speed up the concrete output. As a result, the concrete is of poor quality.

On the other hand, mixing the concrete for a longer period of time is uneconomical in terms of concrete production rate and fuel consumption. As a result, it is critical to mix the concrete for the shortest amount of time possible. The experiments show that the quality of concrete in terms of compressive strength increases with increasing mixing time, but the improvement in compressive strength is not very significant for mixing times greater than two minutes. For our study we took (2) minute as a mixing time.

3.2.1.3 Placing concrete:

It is not enough for a concrete mix to be correctly designed, batched, mixed, and transported; the concrete must also be placed in a systematic manner to yield the best results. For our study we took the same procedure for placing concrete for all the specimens and beams.

3.2.2 Aggregates

Aggregates are inert granular materials such as sand, gravel, or rounded stone that are used in concrete along with water and Portland cement.

Aggregates for a good concrete mix must be clean, hard, and strong particles free of absorbed chemicals or coatings of clay and other fine materials that could cause concrete deterioration. Aggregates, which make up 60 to 75 percent of the total volume of concrete, are classified into two types: fine and coarse. For our study we use rounded aggregate with maximum size of (12.5) mm.

3.2.2.1 Fine Aggregate (Sand)

When aggregate is sieved through a 4.75 mm sieve, the aggregate that passes through it is referred to as fine aggregate. Natural sand is commonly used as fine aggregate, but silt and clay have also been included in this category.

Locally available sand from (Aski- Kalak Source) was used in this experimental work. The sand was clean and with maximum size (4.75 mm), the grading curve of the fine aggregate is shown in the figure 3-1. Also, test results within the lower and upper limit of the specification of ASTM (ASTM-C33) are displayed in table 3-3.

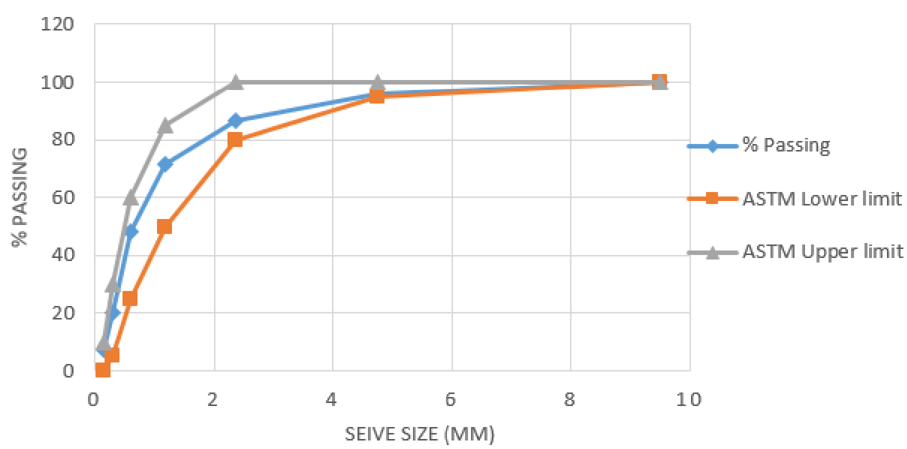

Fig. 3-1 Grading curve for the fine aggregate with ASTM limits

Table 3-3 Grading of the Fine Aggregates (sand)

Sieve size (mm)	Passing %	ASTM Limits	
		Lower	Upper
9.5	100	100	100
4.75	95.77	95	100
2.36	86.59	80	100
1.18	71.44	50	85
0.6	48.1	25	60

0.3	20.32	5	30
0.15	7.17	0	10

3.2.2.2 Course Aggregate

The aggregate retained after sieving through a 4.75 mm sieve is known as coarse aggregate. This category includes gravel, cobble, and boulders. Some conditions may influence the maximum size aggregate used. In general, 40 mm aggregate is used for normal strength concrete and 20 mm aggregate is used for high strength concrete.

The locally available gravel was utilized, the sieve analysis of the aggregates of maximum size (D max = 12.5 mm) are shown in figure 3-2 and the test results within the lower and upper limit of the specification of ASTM (ASTM-C33) as shown in table 3-4.

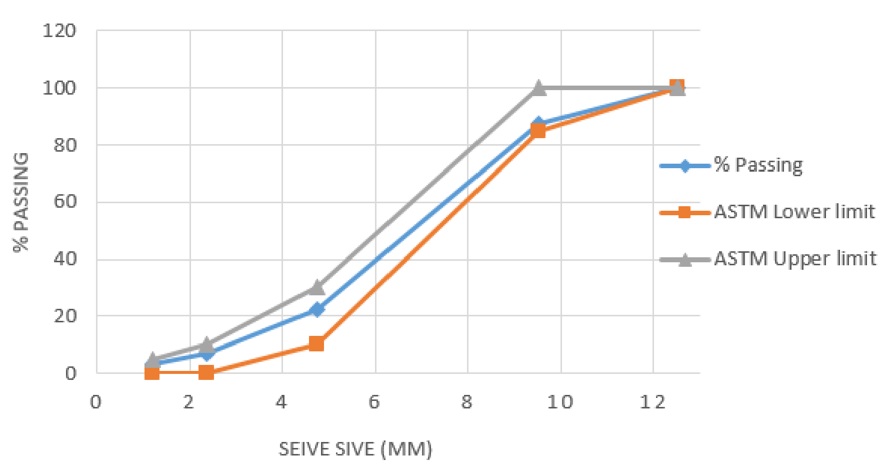

Fig. 3-2 Grading curve for the course aggregate with ASTM limits

Table 3-4 Grading of the Course Aggregates maximum size (D max = 12.5 mm)

Sieve size (mm)	Passing %	ASTM Limits	
		Lower	Upper
12.5	100	100	100
9.5	87.48	85	100
4.75	22.3	10	30
2.36	6.8	0	10
1.18	3.34	0	5

3.2.3 Water

The amount of water in a given volume of concrete has a significant impact on its workability. The greater the water content per cubic meter of concrete, the greater the fluidity of the concrete, which is one of the important factors influencing workability. At the job site, supervisors who are unfamiliar with the practice of making good concrete add more water to increase workability. This practice is frequently used because it is one of the simplest corrective measures that can be implemented on-site. It should be noted that, in terms of desirability, increasing the water content is the last resort for improving workability, even in the case of uncontrolled concrete. If all other steps to improve more water are considered, it is not possible to arbitrarily increase the water content of controlled concrete. More water can be added as long as a corresponding amount of cement is also added to keep the water/cement ratio constant and the strength constant.

In this study, clean potable water was used for concrete casting and curing with water cement ratio (0.4).

3.3 Mix proportion:

The aggregate/cement ratio has a significant impact on workability. The greater aggregate/cement ratio, which lean the concrete. In lean concrete, there is less paste available to provide lubrication per unit surface area of aggregate, limiting aggregate mobility. In the case of rich concrete with a lower aggregate/cement ratio, however, more paste is available to make the mix cohesive and fatty, resulting in better workability.

3.4 Specific gravity of Fine aggregate (Sand)

The specific gravity of an aggregate is used to determine the material's strength or quality. Specific gravity is defined as the weight ratio of a given volume of aggregate to an equal volume of water. Aggregates with low specific gravity are typically weaker than aggregates with high specific gravity. This property aids in the identification of aggregates in general.

the specific gravity results of our test for Fine aggregate is (2.6) and for Course aggregate is (2.65) which are good results and within the range.

3.5 Reinforcing Steel

Deformed steel bars of 10mm and 12mm diameter were used. 12mm diameter bars were used for the bottom main reinforcement, and 10mm diameter bars are used for the top main reinforcement and stirrups. The strength properties of the steel bars have been determined from testing three samples of each type by universal hydraulic tensile test machine (600 KN), as shown in the figure 3-3. The testing results are shown in table 3-5 and figure 3-4.

Fig. 3-3 Tensile testing machine for steel bar

Table 3-5 Properties of reinforcing steel for the experiment

No.	Diameter	L_1 (mm)	f_y (yield) (MPa)	Average f_y (MPa)	f_u (Ultimate) (MPa)	Average f_u Mpa	Elongation (%)
1	10	226	630.6	648.8	733.76	752.04	24
2	10	231	666.2		775.8		29
3	10	230	649.6		746.56		28
4	12	234	636.1	661.43	756.37	783.39	32
5	12	233	644.9		767.87		31
6	12	231	703.3		825.94		29

Fig. 3-4 Steel bar after testing

3.6 Concrete Mix Design:

One of the ultimate goals of researching the different properties of concrete materials, such as plastic concrete and hardened concrete, is to enable a concrete technologist to design a concrete mix for a specific strength. The design of a concrete mix is not an easy task due to the widely varying properties of the constituent materials, the conditions at the work site, and the conditions required for the specific work for which the mix is designed. The design of a concrete mix necessitates comprehensive knowledge of the various properties of these constituent materials, the implications of changes in site conditions, the impact of plastic concrete properties on hardened concrete, and the complicated interrelationship between the variables.

Mix design is the process of selecting appropriate concrete ingredients and determining their relative proportions with the goal of producing concrete with a certain minimum strength and durability as economically as possible.

3.6.1 Concept of Mix Design:

At this point, it is worthwhile to recall the relationships between aggregate and paste, which are the two essential ingredients of concrete. The lubricating effect of the paste provides mass workability, which is influenced by the amount of paste dilution. The paste strength limits the strength of concrete because mineral aggregates, with rare exceptions are far stronger than the paste compound. Because little water flows through aggregate either under pressure or by capillarity, the permeability of concrete is primarily determined by the quality and consistency of the paste. Furthermore, the primary contributor to concrete drying shrinkage is paste. Because the paste governs the properties of concrete to a large extent, it is useful to examine the paste structure more closely. The fresh paste is a suspension, not a cement-in-water solution.

3.6.2 Variables in Proportioning:

The four variable factors to consider when specifying a concrete mix with the given materials are:

(a) Water-cement ratio.

(b) Cement content or cement-aggregate ratio.

(c) Gradation of the aggregates.

(d) Consistency.

In general, none of these four interconnected variables can be chosen or manipulated arbitrarily. Typically, two or three factors are specified, and the others are adjusted to provide the best possible workability and economy. The water/cement ratio expresses paste dilution; cement content varies directly with paste amount. The amount of fine and coarse aggregates used to control aggregate gradation. The practical requirements of placement establish consistency.

3.6.3 Mix Design:

According to American Concrete Institute (ACI) 318M-14 recommendations, many trial mixes for concrete with maximum aggregate size (12.5) were designed to obtain a compressive strength of (35 MPA). The final mix proportions were used for the beams shown in table 3-6, but trial mixes are required because the relationships between concrete strength and water-cement ratio depend to some extent on the local aggregates and cement sources. Experience with the quality and characteristics of both cement and aggregates in a specific location can be extremely valuable during the mix design stage.

1- Water/cement ratio (w/c) = 0.4
2- Water content 220 kg/m3
3- Cement content 545 kg/m3
4- Total aggregate content 1635 kg/m3
5- Mix proportions per cubic meter.

Table 3-6 The ingredients of the concrete mix design for one beam

Max. Agg. size D max (mm)	Cement (kg)	Sand (kg)	Course Agg. (kg)	w/c ratio	Mix design ratio	Average compres-sive Strength f_c' (MPa)	C/Agg.
12.5	25.546	31.92	44.718	0.4	1:1.25: 1.75	35	0.57

To achieve a homogeneous mix, a mechanical mixer was used. In the study, twenty-eight mix batches were tested. The research looks at the effects of vibration, delayed vibration, and repeated vibration at various times.

For mix design we produce a mix with ratio as mentioned in table 3-6 and the samples were tested in 28 days and the result were a range of 35 MPa, but because of our laboratory was not prepared in this time for this reason all the beams and cylinders of our tests done after 56 days.

For each mix, the following elements were casted:
(1) Beams 1500 x 125 x 250 mm to be tested at 56 days.
(2) Cylinders of 150 mm diameter and 300 heights will be tested at 56 days,
In standard mix design form, the relevant details and calculations are shown.

3.7 Beam Formworks

3.7.1 Molds

To assess the re-vibrated concrete's plastic shrinkage cracking, plywood block sheets of dimensions (1220x440x18) mm were used. The molds were created from these plywood block sheets and the size of the beam formworks are (125 x 250 x 1500mm). Figure 3-5 shows how the molds were cleaned and given a light oil coating to make removal of the specimen easier.

Fig. 3-5 Molds of the tested specimens

3.8 Cages and Reinforced Placement

For our test twenty- four beams were reinforced to the same reinforcement ratio. bars The beams for group (A, B, C, D, E and F) reinforced with longitudinal top reinforcement of (2ø10mm) and bottom reinforcement of (2ø12mm) with trans-vers reinforcement (Stirrups) ø12mm which spread all over the beams as shown in figure 3-6 except group F which are without transverse reinforcement and group G is without longitudinal and transverse reinforcement, the total beam shown in figure 3-7.

Fig. 3-6 Reinforcement cages and molds

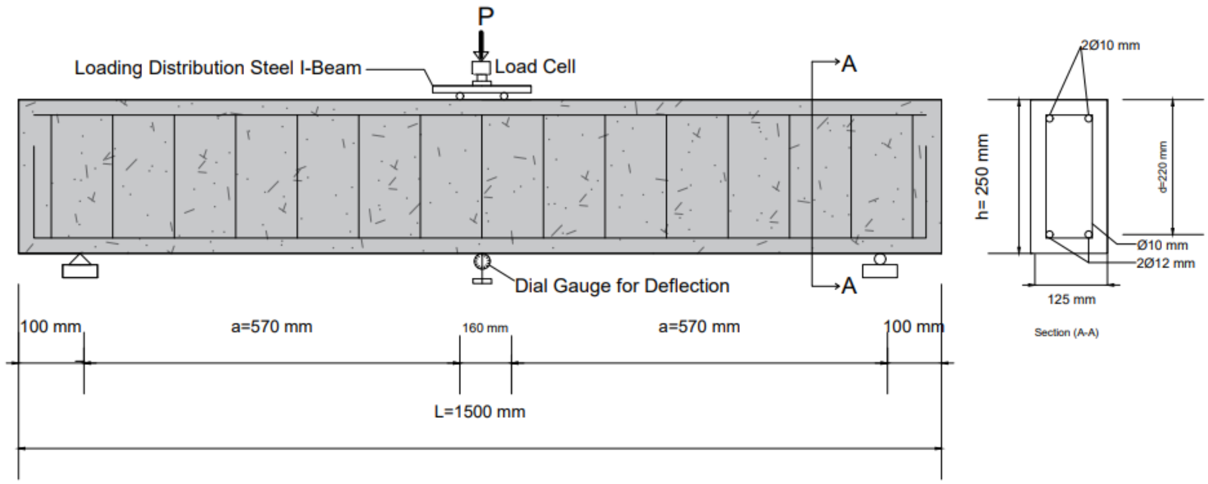

Fig. 3-7 Longitudinal and transverse cross section view

3.9 Casting of Concrete

All the beams and cylindrical specimens were casted manually after the molds were cleaned and oiled. The reinforcement bars were placed carefully inside the mold, after that the concrete placed in layers inside the molds. Six cylinders of (150 x 300) mm were casted for each beam to determine the mechanical properties of concrete compressive strength, splitting tensile strength at age 56 days were determined. Figure 3-8 shows the beams and control specimens after casting.

After casting the concrete into the beam and cylindrical molds, an external table vibration as shown in figure 3-9 was applied to the specimens with different time intervals from 15 to 60 seconds. After the first vibration, concrete beams and cylinders were taken of and waited for the second vibration with different waiting time (30, 60, 90 and 120) minutes.

During this time, the concrete hardened and water continued to bleed to the beams surfaces. After the hardening process had finished, the specimens were subjected to a second vibration. The following vibration times were used for the second vibration: (15, 30, 45, and 60) seconds which called re-vibration. The concrete beams and cylinders were screed perpendicular to the stress risers after the second vibration. A smooth steel trowel was used to smooth out the concrete surface.

Fig. 3-8 Casting Specimens

Fig. 3-9 Electrical vibration of specimen

3.10 Curing

The casted samples were removed from the molds after 24 hours, and they were cured for 28 days by covering the beams and control specimens with burlap and kept moist during the curing period. According to ASTM C31for 24 hours, as shown in the figure 3-10.

Fig. 3-10 Curing of beams and cylinders

3.11 Concrete Mechanical Properties

3.11.1 Compressive Strength:

The compressive strength of a concrete cylinder is one of the most common performance measures used by structural engineers. The compressive strength of concrete cylinders is determined here by continuously applying load to the cylinder until failure occurs. The test is conducted on a compression-testing machine. Although according to ASTM C39, three cylinders (150 x 300) were tested for each beam. According to the ASTM C617 cylinder, capping was required to give a smooth and level surface for applying a compressive load to concrete cylinders. The stone powder method was used for cylinder capping, as shown in figure 3-11.

Fig. 3-11 Cylinder capping concrete

3.12 Splitting Tensile Strength (f_{sp})

The tensile strength of concrete is a fundamental and important property that influences the extent and size of cracking in structures. Furthermore, due to its brittle nature, the concrete is very weak in tension. As a result, it is not expected to withstand direct tension. Concrete cracks as tensile forces exceed its tensile strength. As a result, determining the tensile strength of concrete is required to determine the load at which the concrete members may crack.

Three (150 x 300) mm cylinders were tested for each beam to determine the tensile strength of concrete. Furthermore, a splitting tensile strength test on a concrete cylinder is a method for determining concrete tensile strength. The tests were performed according to ASTM (ASTM-C469). Furthermore, the ALFA testing equipment with the capacity of 2000 NK was used for the performance of the splitting tensile test, as shown in figure 3-12.

Fig. 3-12 ALFA Testing Equipment

3.13 Experimental Program

The experimental program of this research contains twenty- eight beams for testing, the concrete beams with transverse reinforced (stirrups) spread all over the beam with constant compressive strength, except group F which are without transverse reinforcement (stirrups) and group G without longitudinal and transverse reinforcement. The beams cross-section dimensions are (125 x 250 x 1500) mm, the maximum aggregate size is (12.5) mm were used in the experimental work. The beams were divided into seven groups (A B, C, D E, F and G) as shown in table 3-7, and each group had four specimen beams.

The beams for group (A, B, C, D, E and F) reinforced with longitudinal top reinforcement of (2ø10mm) and bottom reinforcement of (2ø12mm) with transvers reinforcement (Stirrups) ø12mm with (10) cm equally c.c. spacing all over the beams except group F which are without transverse reinforcement and group G is without longitudinal and transverse reinforcement. And 144 cylinders (300 mm height and 150 mm diameter) were used for testing the compressive and tensile strength of concrete. These sampled were tested after 56 days in attempt to study the effects of vibration, delay in re-vibration and time duration of vibration on the development of concrete strength.

Based on the findings of Vollick (1958), Aldalinsi et al. (2003), and Krishna et al. (2008), the specimens were vibrated for time intervals of (15, 30 ,45, and 60) seconds, and the waiting time after specimen casting and before a second vibration (re-vibration time lag) was chosen as (30, 60, 90, and 120) minutes. To assess the effect of the length of the second vibration, four re-vibration times (15, 30, 45, and

60) seconds were chosen. These re-vibration time lengths resulted in good vibration of the mixtures, as evidenced by the mix's uniformity and the visual sign of good compaction.

Table 3-7 Experimental program of beam specimens

No.	Group	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)
1	With Stirrups	A1	15	0	0
2		A2	30	0	0
3		A3	45	0	0
4		A4	60	0	0
5	With Stirrups	B1	15	30	15
6		B2	15	30	30
7		B3	15	30	45
8		B4	15	30	60
9	With Stirrups	C1	15	60	15
10		C2	15	60	30
11		C3	15	60	45
12		C4	15	60	60
13	With Stirrups	D1	15	90	15
14		D2	15	90	30
15		D3	15	90	45
16		D4	15	90	60
17	With Stirrups	E1	15	120	15
18		E2	15	120	30

19		E3	15	120	45
20		E4	15	120	60
21	without Stir-rups	F1	15	60	15
22		F2	15	60	30
23		F3	15	60	45
24		F4	15	60	60
25	without rein-forcement	GI	15	60	15
26		G2	15	60	30
27		G3	15	60	45
28		G4	15	60	60

3.14 Loading Setup

The beams were tested in the College of Engineering at Tishk International University in (Civil Engineering Laboratory). At the age of 56 days all the beams were tested. When the curing was completed, the specimens were placed at the laboratory temperature and were painted with white color and drawing the gridlines over the beam, so the cracks could be seen quickly and clearly and the crack pattern could be marked, as shown in figure 3-13. For installing the strain gauge over the beam first able we polish the surface of the beam by using diamond angle grinder disc to obtain smooth surface, then we place the strain gauge over a clean glass surface and we cover the strain gauge with an adhesive tape, the strain gauge and the surface are now ready for the bonding process. For this process we use super glue Loctite bonding materials that was set few seconds after application. After that we put a metal load over the stain gauge for 24 hours till it completely hardened. The load was applied over the beam on the two-point line load with a distance of 16 cm. The applied load was calculated with digital readout as shown in the figure 3-14, and we connect the stain gauge to data logger for reading the maximum strain. And we use a dial gauge that had the smallest count of 0.01mm for measuring the displacements that was placed at the center of the beam to determine the center displacement, as illustrated in figure 3-13. The loading process for all the beams were videoed for the purpose of recording crack behavior of the beams.

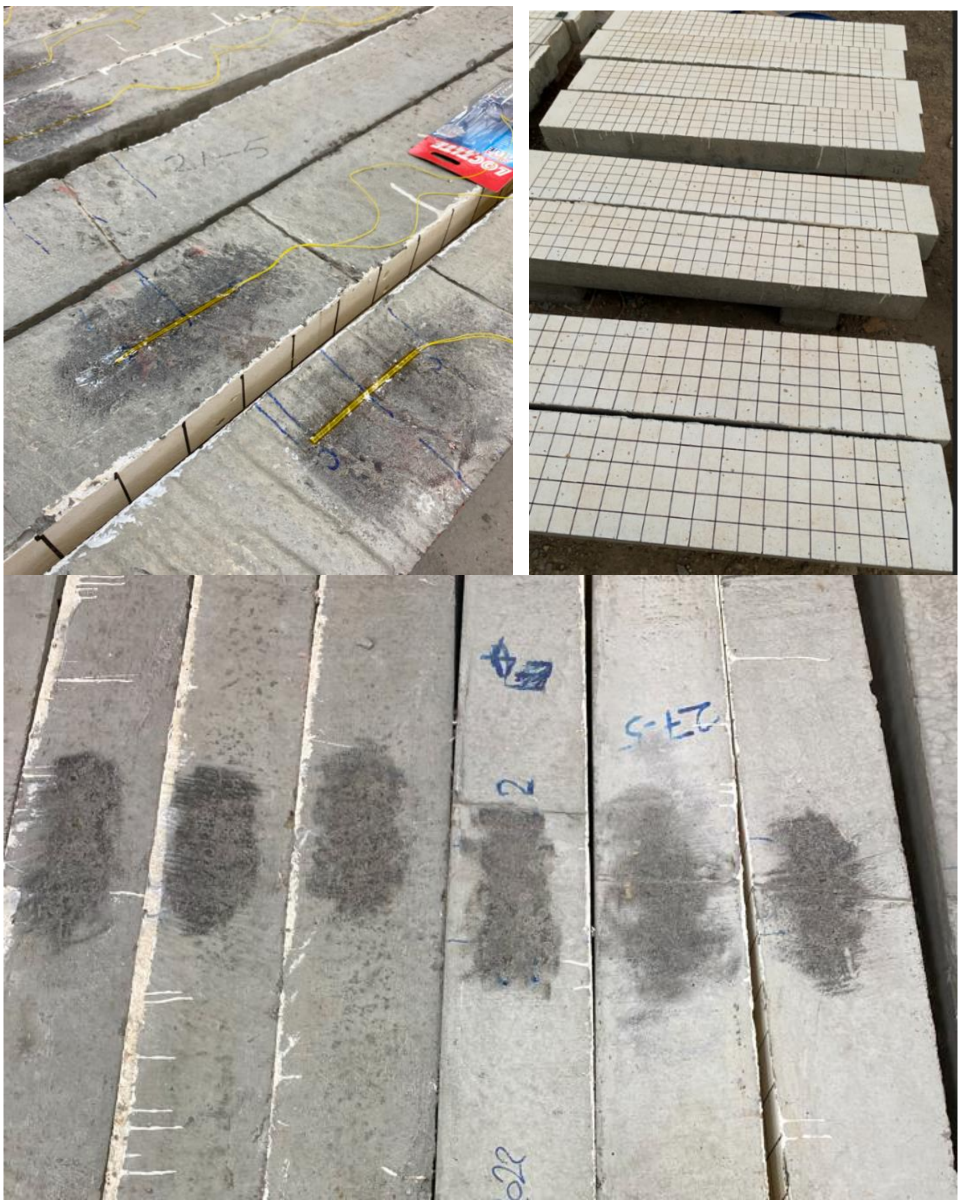

Fig. 3-13 Beam Specimens painted with a white color and smoothed the strain gauge area

Fig. 3-14 Test setup with the loading frame

CHAPTER FOUR

RESULTS AND DISCUYSSION

4.1 Introduction

The experimental results of twenty-eight beams were presented through this chapter in many figures and tables to explore and show the effect of re-vibration of beams on the mechanical properties of concrete, behavior of reinforced concrete beam, modes of failure load, first crack load, strain and maximum deflection of the beams, behavior of each beam are described in this chapter. Presenting the detailed data, calculations, analysis, and working photos.

4.2Properties of Control Specimens

4.2.1 Concrete Compressive Strength

According to ASTM C39, three cylinders (150 x 300) were tested for each beam as shown in table 4-1. After testing and according to the ASTM C617 cylinder, table 4-1 shows the concrete compressive strength of all beams, capping was required to give a smooth and level surface for applying a compressive load to concrete cylinders.

To analyze the table and identify the beam with the maximum compressive strength in each group, we can look at the average compressive strength values for each specimen. Figure 4-1 provides information on the compressive strength of different specimens under varying vibration and waiting conditions. The analysis allows us to determine the specimen with the maximum compressive strength in each group, aiding in understanding the impact of different parameters on the strength of the beams,

Which shows that the maximum compressive strength of concrete f_c' at re-vibration time (45 sec.) for all waiting time 30, 60, 90, and 120 sec. , this mean that important of re-vibration time in re-compacting the concrete.

Table 4-1 Compressive strengths of concrete for all concrete specimens

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	f_c' (1) (MPa)	f_c' (2) (MPa)	f_c' (3) (MPa)	Average f_c' (MPa)	$\frac{f_c'}{f_{c,o}'}$
1	A1	15	0	0	32.52	32.95	33.15	32.873	1
2	A2	30	0	0	33.84	34.22	34.54	34.2	1.04
3	A3	45	0	0	35.11	35.61	36.79	35.836	1.09
4	A4	60	0	0	31.24	30.74	30.88	30.953	0.95
5	B1	15	30	15	35.66	34.23	36.78	35.556	1.09
6	B2	15	30	30	36.95	37.25	37.44	37.213	1.14
7	B3	15	30	45	38.96	37.55	38.43	38.313	1.17
8	B4	15	30	60	34.32	33.68	33.25	33.75	1.03
9	C1	15	60	15	38.22	38.94	39.78	38.98	1.19
10	C2	15	60	30	40.11	39.77	39.42	39.766	1.21

11	C3	15	60	45	42.14	43.75	43.88	43.256	1.32
12	C4	15	60	60	37.79	38.15	38.66	38.2	1.17
13	D1	15	90	15	36.67	35.22	36.22	36.036	1.10
14	D2	15	90	30	38.2	37.21	38.63	38.013	1.16
15	D3	15	90	45	39.16	38.52	38.98	38.886	1.19
16	D4	15	90	60	34.23	33.15	34.41	33.93	1.04
17	E1	15	120	15	31.22	32.51	31.78	31.836	0.97
18	E2	15	120	30	33.28	34.66	33.17	33.703	1.03
19	E3	15	120	45	34.54	35.6	35.78	35.30	1.07
20	E4	15	120	60	30.43	30.78	31.17	30.793	0.94
21	F1	15	60	15	38.22	38.94	39.78	38.98	1.19
22	F2	15	60	30	40.11	39.77	39.42	39.766	1.21
23	F3	15	60	45	42.14	43.75	43.88	43.256	1.32
24	F4	15	60	60	37.79	38.15	38.66	38.2	1.17
25	G1	15	60	15	38.22	38.94	39.78	38.98	1.19
26	G2	15	60	30	40.11	39.77	39.42	39.766	1.21
27	G3	15	60	45	42.14	43.75	43.88	43.256	1.32
28	G4	15	60	60	37.79	38.15	38.66	38.2	1.17

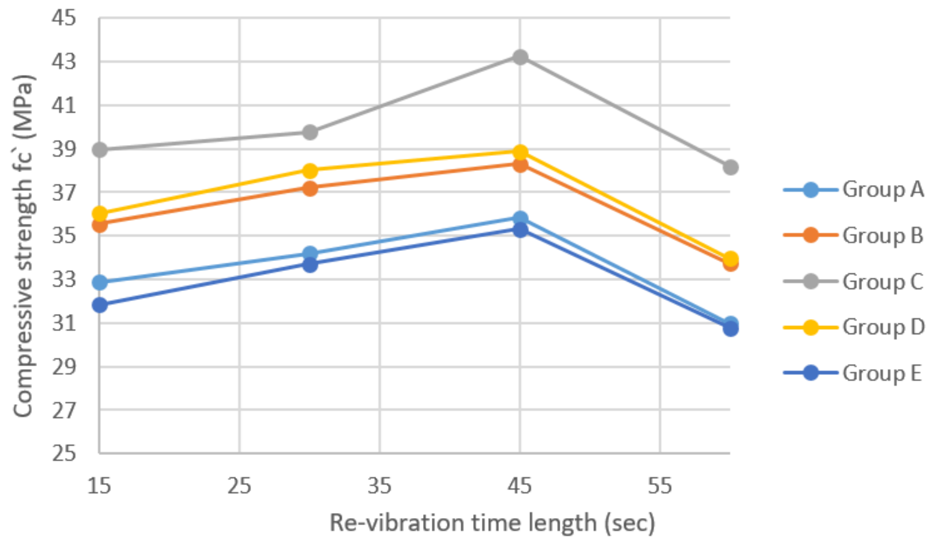

Fig. 4-1 Average compressive strengths of concrete for initial vibration and re-vibrated concrete specimens with variable time length

Table 4-2 and figure 4-2 shows the effect of waiting time after initial vibration, for re-vibration time and the percentage relationship between the parameters and the compressive strength, we can compare the changes in compressive strength while keeping certain parameters constant. Let's focus on the "Waiting time" parameter as it varies across the specimens. The results show that at waiting time (30 min.) the compressive strength increased by about (9%) and then at waiting time (60 min.) the increasing in compressive strength became maximum amount (19 %), but after this time at waiting time (90 and 120 min.) the compressive strength de-creased to (10 %) and (-3 %) respectively, this mean that the optimum waiting time is (60 min.), to obtain the maximum compressive strength of concrete, this mean that the waiting time should be in the interval of initial setting time of the cement.

After this time the concrete began in the interval of final setting and any re-vibra-tion of the concrete gives negative effects and making cracks after hardening of the concrete, depending on this reason the concrete strength is reduced.

Table 4-2 Compressive strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f'_c (MPa)	$\frac{f'_c}{f'_{c,o}}$
1	A1	15	0	0	32.873	1
2	B1	15	30	15	35.556	1.09
3	C1	15	60	15	38.98	1.19
4	D1	15	90	15	36.036	1.10
5	E1	15	120	15	31.836	0.97

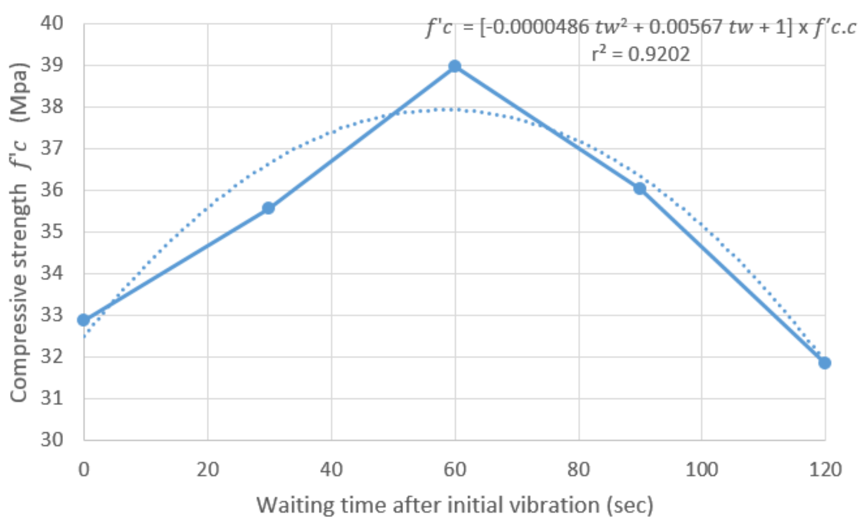

Fig. 4-2 Compressive strengths of concrete with re-vibration time (15 sec) with different waiting time

Table 4-3 and figure 4-3 shows the effect of waiting time after initial vibration, for re-vibration time and understand the percentage relationship between the parameters (initial vibration time, waiting time after initial vibration, re-vibration time) and the compressive strength, The initial vibration time for all the specimens is fixed at 15 seconds with variable waiting time after the initial vibration 30, 60, 90 and 120 min, By comparing the compressive strength values, we can observe the percentage change in compressive strength with respect to the waiting time after the initial vibration.

The results show that at waiting time (30 min.) the compressive strength increased by about (13%) and then at waiting time (60 min.) the increasing in compressive strength became maximum amount (21 %), but after this time at waiting time (90 and 120 min.) the compressive strength decreased to (16 %) and (3 %) respectively, this mean that the optimum waiting time is (60 min.).

To obtain the maximum compressive strength of concrete, this mean that the waiting time should be in the interval of initial setting time of the cement. After this time the concrete began in the interval of final setting time and re-vibration of the concrete gives minimum effects and making cracks after hardening of the concrete depending on this reason the concrete strength is reduced.

Table 4-3 Compressive strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f_c' (MPa)	$\frac{f_c'}{f_{c,o}'}$
1	A1	15	0	0	32.873	1
2	B2	15	30	30	37.213	1.13
3	C2	15	60	30	39.766	1.21
4	D2	15	90	30	38.013	1.16
5	E2	15	120	30	33.703	1.03

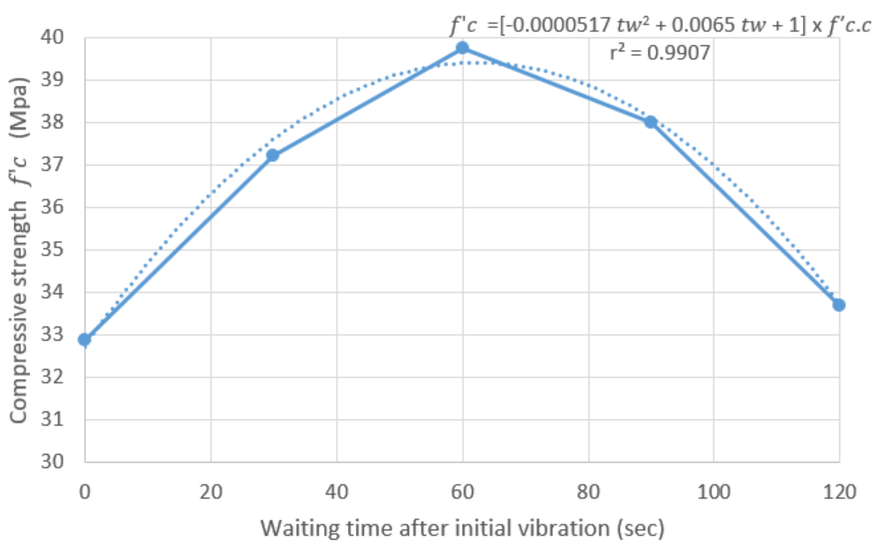

Fig. 4-3 Compressive strengths of concrete with re-vibration time (30 sec) with different waiting time

Table 4-4 and figure 4-4 shows relationship between the parameters (initial vibration time, waiting time after initial vibration, re-vibration time) and the compressive strength, the initial vibration time for all the specimens is fixed at 15 seconds, so there is no variation in this parameter with variable waiting time after the initial vibration and all the specimens have a re-vibration time of 45 seconds. By comparing the compressive strength values, we can observe the percentage change in compressive strength with respect to the waiting time after the initial vibration.

The results show that at waiting time (30 min.) the compressive strength increased by about (17%) and then at waiting time (60 min.) the increasing in compressive strength became maximum amount (32 %), but after this time at waiting time (90 and 120 min.) the compressive strength decreased to (18 %) and (7 %) respectively, this mean that the optimum waiting time is (60 min.).

It's important to note that this analysis focuses on the waiting time parameter only. To comprehensively understand the relationship between all the parameters and compressive strength, further analysis considering each parameter's effect individually and in combination would be necessary.

To obtain the maximum compressive strength of concrete, this mean that the waiting time should be in the interval of initial setting time of the cement. After this time the concrete began in the interval of final setting time and re-vibration of the concrete gives minimum effects and making cracks after hardening of the concrete depending on this reason the concrete strength is reduced.

Table 4-4 Compressive strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibra-tion time (sec)	Waiting time after initial vibration (min)	Re-vibra-tion time (sec)	Compressive strength f_c' (MPa)	$\frac{f_c'}{f_{c,o}'}$
1	A1	15	0	0	32.873	1
2	B3	15	30	45	38.313	1.17
3	C3	15	60	45	43.256	1.32
4	D3	15	90	45	38.886	1.18
5	E3	15	120	45	35.30	1.07

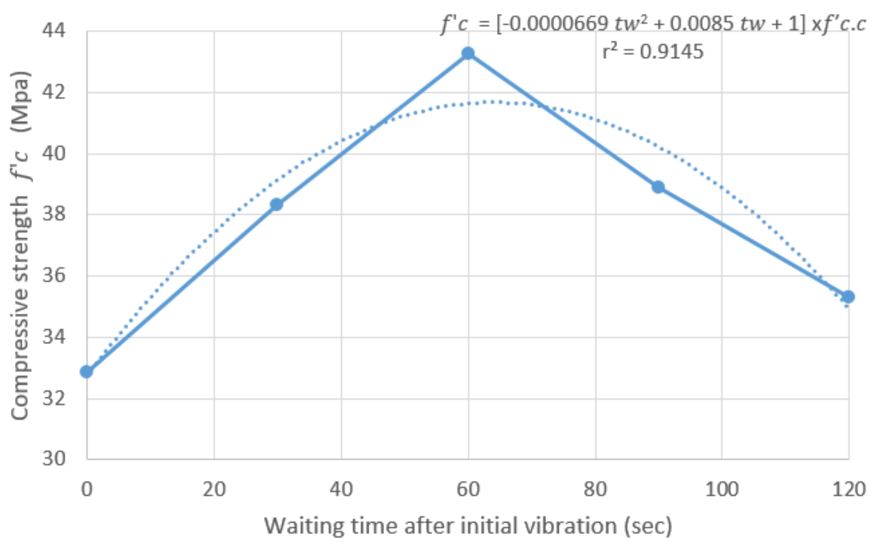

Fig. 4-1 Compressive strengths of concrete with re-vibration time (45 sec) with different waiting time

Table 4-5 and figure 4-5 shows the relationship between the parameters (initial vibration time, waiting time after initial vibration, re-vibration time) and the compressive strength, the initial vibration time for all the specimens is fixed at 15 seconds with variable waiting time after the initial vibration (30, 60, 90 and 120 min.). All the specimens have a re-vibration time of 60 seconds. By comparing the compressive strength values, we can observe the relationship between the waiting time after the initial vibration and the compressive strength, The results show that at waiting time (30 min.) the compressive strength increased by about (3 %) and then at waiting time (60 min.) the increasing in compressive strength became maximum amount (16 %), but after this time at waiting time (90 and 120 min.) the compressive strength decreased to (3 %) and (-6 %) respectively, this mean that the optimum waiting time is (60 min.).

This analysis specifically considers the waiting time parameter. To gain a comprehensive understanding of the correlation between all the parameters and compressive strength, a more extensive analysis is required, taking into account the individual and combined effects of each parameter.

Table 4-5 Compressive strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f'_c (MPa)	$\frac{f'_c}{f'_{c,o}}$
1	A1	15	0	0	32.873	1
2	B4	15	30	60	33.75	1.03
3	C4	15	60	60	38.2	1.16

4	D4	15	90	60	33.93	1.03
5	E4	15	120	60	30.793	0.94

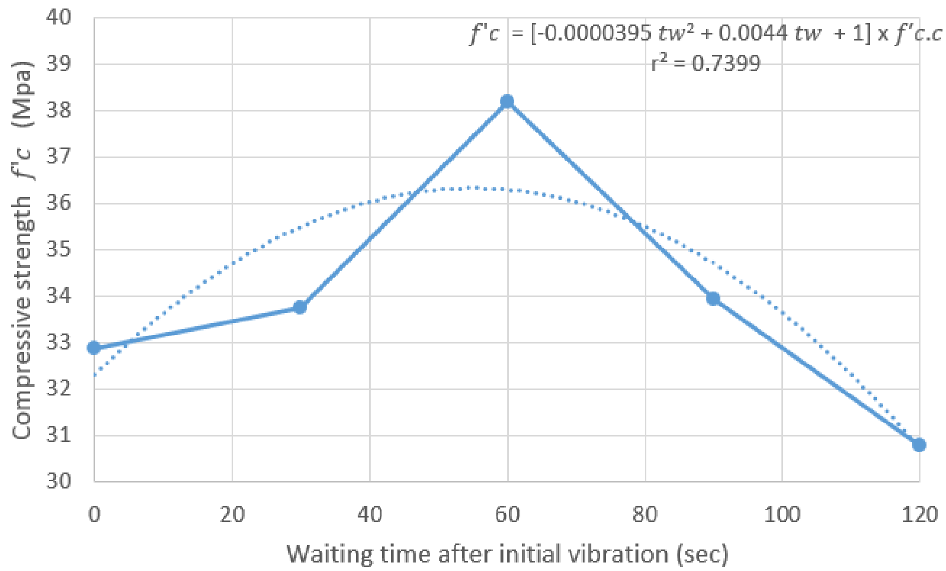

Fig. 4-5 Compressive strengths of concrete with re-vibration time (60 sec) with different waiting time

4.2.2 Splitting Tensile Strength

According to ASTM (ASTM-C469), For each beam, three (150 x 300) mm cylinders were tested to determine the splitting tensile strength of concrete (f_{sp}), Table 4-6 shows the results of splitting tensile strength of concrete.

To analyze the table 4-6 and examine the splitting tensile strength, we can focus on the provided data. Figure 4-6 provides information on the splitting tensile strength of different specimens under varying vibration and waiting conditions.

Which shows that the maximum splitting tensile strength of concrete f_{sp} at re-vibration time (45 sec.) for all waiting time 30, 60, 90, and 120 min. , this mean that important of re-vibration time in re-compacting the concrete. From this analysis, we can observe the variation in splitting tensile strength across the specimens. Specimens C3, G3, and F3 have the highest splitting tensile strength of 1.35 MPa, while Specimen E4 has the lowest splitting tensile strength of 0.84 MPa. The other specimens exhibit values ranging between 0.95 MPa and 1.2 MPa.

It's important to note that the splitting tensile strength indicates the ability of the material to resist cracking or splitting under tensile stress. The variation in splitting tensile strength among the specimens can be attributed to differences in their composition, curing conditions, or other factors that influence the material's behavior. Splitting Tensile Strength equation

$$f_{sp} = \frac{2P}{\pi Ld}$$

Where:

f_{sp} = splitting tensile strength (MPa)

P = maximum applied load (KN)

L = length (mm)

d = depth (mm)

Table 4-6 Splitting tensile strengths of for all concrete specimens

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Failure load kN	Failure load kN	Failure load kN	Average failure load kN	Splitting tensile strength f_{sp} (MPa)	$\frac{f_{sp}}{f_{sp,o}}$
1	A1	15	0	0	234.848	200.233	249.475	228.185	3.22	1
2	A2	30	0	0	233.216	263.201	246.41	247.609	3.5	1.09
3	A3	45	0	0	206.696	298.101	263.201	255.999	3.62	1.13
4	A4	60	0	0	224.887	210.761	212.669	216.105	3.05	0.95
5	B1	15	30	15	229.751	250.973	247.365	242.696	3.43	1.07
6	B2	15	30	30	251.141	281.496	226.553	253.063	3.58	1.12
7	B3	15	30	45	273.954	241.346	253.406	256.235	3.62	1.13
8	B4	15	30	60	202.498	222.088	249.608	224.731	3.17	0.99
9	C1	15	60	15	280.639	274.995	234.549	263.394	3.72	1.16

10	C2	15	60	30	261.412	287.629	267.466	272.169	3.85	1.20
11	C3	15	60	45	322.54	310.755	287.893	307.062	4.34	1.35
12	C4	15	60	60	260.136	210.228	230.315	233.559	3.3	1.03
13	D1	15	90	15	235.282	232.75	203.231	223.754	3.16	0.99
14	D2	15	90	30	249.008	227.419	225.753	234.06	3.31	1.03
15	D3	15	90	45	241.489	252.673	266.999	253.720	3.58	1.12
16	D4	15	90	60	230.951	157.321	266.066	218.112	3.08	0.96
17	E1	15	120	15	222.088	205.416	223.539	217.014	3.07	0.96
18	E2	15	120	30	201.365	235.415	247.394	228.058	3.22	1
19	E3	15	120	45	251.027	251.207	226.834	243.022	3.43	1.07
20	E4	15	120	60	194.969	187.572	189.771	190.770	2.69	0.84
21	F1	15	60	15	280.639	274.995	234.549	263.394	3.72	1.16
22	F2	15	60	30	261.412	287.629	267.466	272.169	3.85	1.2
23	F3	15	60	45	322.54	310.755	287.893	307.062	4.34	1.35

24	F4	15	60	60	260.136	210.228	230.315	233.559	3.3	1.03
25	G1	15	60	15	280.639	274.995	234.549	263.394	3.72	1.16
26	G2	15	60	30	261.412	287.629	267.466	272.169	3.85	1.2
27	G3	15	60	45	322.54	310.755	287.893	307.062	4.34	1.35
28	G4	15	60	60	260.136	210.228	230.315	233.559	3.3	1.03

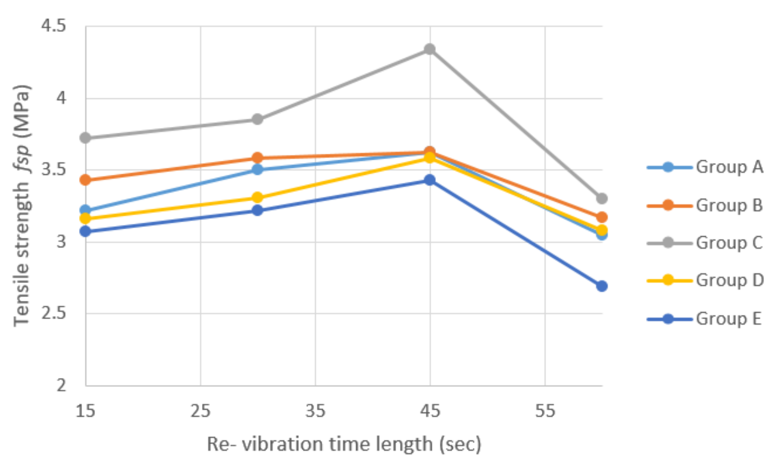

Fig. 4-6 Average tensile strengths of concrete for initial vibration and re-vibrated concrete specimens with time length

Table 4-7 and figure 4-7 shows relationship between the parameters and the splitting tensile strength in the table, we can compare the values of the splitting tensile strength for each specimen.

The results show that at waiting time (30 min.) the splitting tensile strength increased by about (7 %) and then at waiting time (60 min.) the increasing in splitting tensile strength became maximum amount (16 %), but after this time at waiting time (90 and 120 min.) the splitting tensile strength decreased to (-2 %) and (-5 %) respectively, this mean that the optimum waiting time is (60 min.).

From this analysis, we can see the percentage difference in splitting tensile strength compared to the control value. Specimens A1, B1, D1, and E1 have lower splitting tensile strengths than the maximum value, with percentage differences ranging from -2 % to 7 %. Specimen C1 has the maximum splitting tensile strength and serves as the reference with a percentage difference of 16 %.

Overall, this analysis provides a perspective on the relative variations in splitting tensile strength among the specimens, highlighting their percentage differences with respect to the specimen with the maximum value.

To obtain the maximum splitting tensile strength of concrete, this mean that the waiting time should be in the interval of initial setting time of the cement.
After this time the concrete began in the interval of final setting and any re-vibration of the concrete gives negative effects and making cracks after hardening of the concrete, depending on this reason the concrete strength is reduced.

Table 4-7 Splitting tensile strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Splitting tensile strength f_{sp} (MPa)	$\frac{f_{sp}}{f_{sp,o}}$
1	A1	15	0	0	3.22	1
2	B1	15	30	15	3.43	1.07
3	C1	15	60	15	3.72	1.16
4	D1	15	90	15	3.16	0.98
5	E1	15	120	15	3.07	0.95

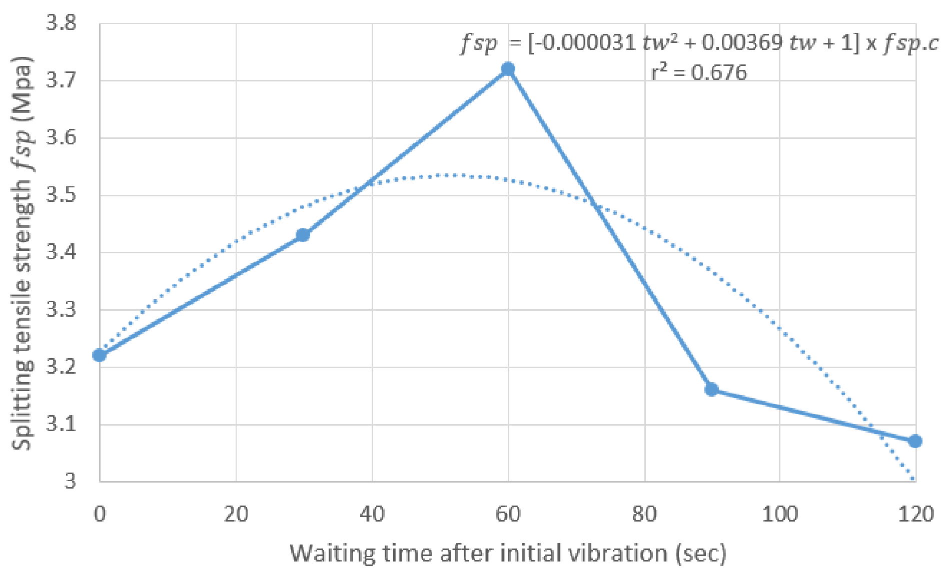

Fig. 4-7 Split tensile strengths of concrete with re-vibration time (15 sec) with different waiting time

Table 4-8 and figure 4-8 shows relationship between the parameters and the splitting tensile strength in the table, we can examine the values and calculate the percentage difference from the maximum splitting tensile strength.

Specimen A1 has an initial vibration time of 15 seconds, no waiting time, and no re-vibration. Its splitting tensile strength is 3.22 MPa.

Specimen B2, C2, D2, and E2 have an initial vibration time of 15 seconds, with variable waiting time of 30, 60, 90 and 120 min. and all have re-vibration time of 30 seconds.

The results show that at waiting time (30 min.) the splitting tensile strength increased by about (11 %) and then at waiting time (60 min.) the increasing in splitting tensile strength became maximum amount (20 %), but after this time at waiting time (90 and 120 min.) the splitting tensile strength decreased to (3 %) and (0 %) respectively, this mean that the optimum waiting time is (60 min.).

From this analysis, we can observe the percentage differences in splitting tensile strength compared to the maximum value. Specimens A1, B2, D2, and E2 have lower splitting tensile strengths than Specimen C2 (the maximum value). Overall, this analysis provides insights into the relative variations in splitting tensile strength among the specimens, highlighting their percentage differences with respect to the specimen with the control value.

Table 4-8 Splitting tensile strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Splitting tensile strength f_{sp} (MPa)	$\frac{f_{sp}}{f_{sp,o}}$
1	A1	15	0	0	3.22	1
2	B2	15	30	30	3.58	1.11
3	C2	15	60	30	3.85	1.20
4	D2	15	90	30	3.31	1.03
5	E2	15	120	30	3.22	1

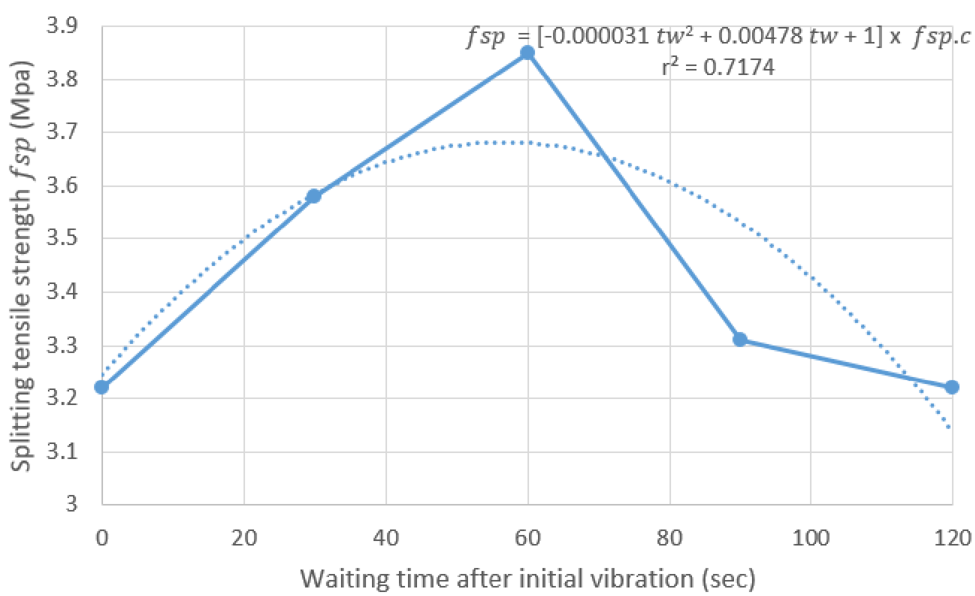

Fig. 4-8 Split tensile strengths of concrete with re-vibration time (30 sec) with different waiting time

Table 4-9 and figure 4-9 shows and compare the percentage relationship between the parameters and the splitting tensile strength in the table, we can calculate the percentage difference from the control splitting tensile strength.

Specimen A1 has an initial vibration time of 15 seconds, no waiting time, and no re-vibration. Its splitting tensile strength is 3.22 MPa.

Specimen B3, C3, D3, and E3 have an initial vibration time of 15 seconds, with variable waiting time of 30, 60, 90 and 120 min. and all have re-vibration time of 45 seconds.

The results show that at waiting time (30 min.) the splitting tensile strength increased by about (12 %) and then at waiting time (60 min.) the increasing in splitting tensile strength became maximum amount (35 %), but after this time at waiting time (90 and 120 min.) the splitting tensile strength decreased to (11 %) and (7 %) respectively, this mean that the optimum waiting time is (60 min.).

From this analysis, we can observe the percentage differences in splitting tensile strength compared to the maximum value. Specimens A1, B3, D3, and E3 have lower splitting tensile strengths than Specimen C3 (the maximum value), this analysis provides insights into the relative variations in splitting tensile strength among the specimens, highlighting their percentage differences with respect to the specimen with the control value.

Table 4-9 Splitting tensile strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Splitting tensile strength f_{sp} (MPa)	$\frac{f_{sp}}{f_{sp,o}}$
1	A1	15	0	0	3.22	1
2	B3	15	30	45	3.62	1.12
3	C3	15	60	45	4.34	1.35
4	D3	15	90	45	3.58	1.11
5	E3	15	120	45	3.43	1.07

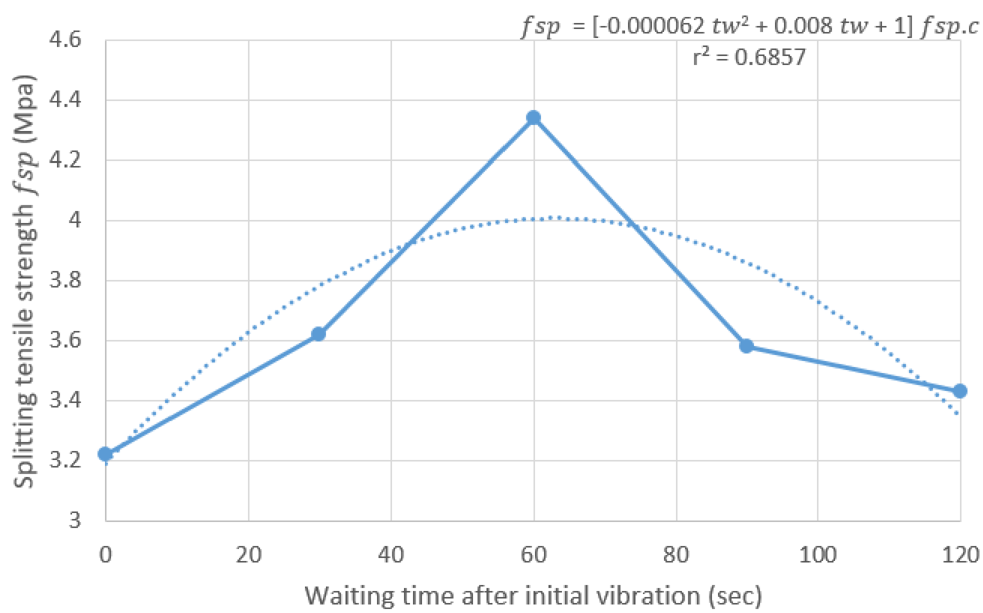

Fig. 4-9 Split tensile strengths of concrete with re-vibration time (45 sec) with different waiting time

Table 4-10 and figure 4-10 shows and compare the percentage relationship between the parameters and the splitting tensile strength in the table, we can calculate the percentage difference from the control splitting tensile strength.

Specimen A1 has an initial vibration time of 15 seconds, no waiting time, and no re-vibration. Its splitting tensile strength is 3.22 MPa.

Specimen B4, C4, D4, and E4 have an initial vibration time of 15 seconds, with variable waiting time of 30, 60, 90 and 120 min. and all have re-vibration time of 60 seconds.

The results show that at waiting time (30 min.) the splitting tensile strength decreased by about (-2 %) and then at waiting time (60 min.) the increasing in splitting tensile strength became maximum amount (2 %), but after this time at waiting time (90 and 120 min.) the splitting tensile strength decreased to (-4 %) and (-16 %) respectively, this mean that the optimum waiting time is (60 min.).

From this analysis, we can observe the percentage differences in splitting tensile strength compared to the maximum value. Specimens A1, B4, D4, and E4 have lower splitting tensile strengths than Specimen C4 (the maximum value). This analysis offers valuable information about the relative differences in splitting tensile strength among the specimens, emphasizing the percentage variances in relation to the specimen with the reference value.

Table 4-10 Splitting tensile strengths of concrete specimens with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Splitting tensile strength f_{sp} (MPa)	$\frac{f_{sp}}{f_{sp,o}}$
1	A1	15	0	0	3.22	1
2	B4	15	30	60	3.17	0.98
3	C4	15	60	60	3.3	1.02
4	D4	15	90	60	3.08	0.96
5	E4	15	120	60	2.69	0.84

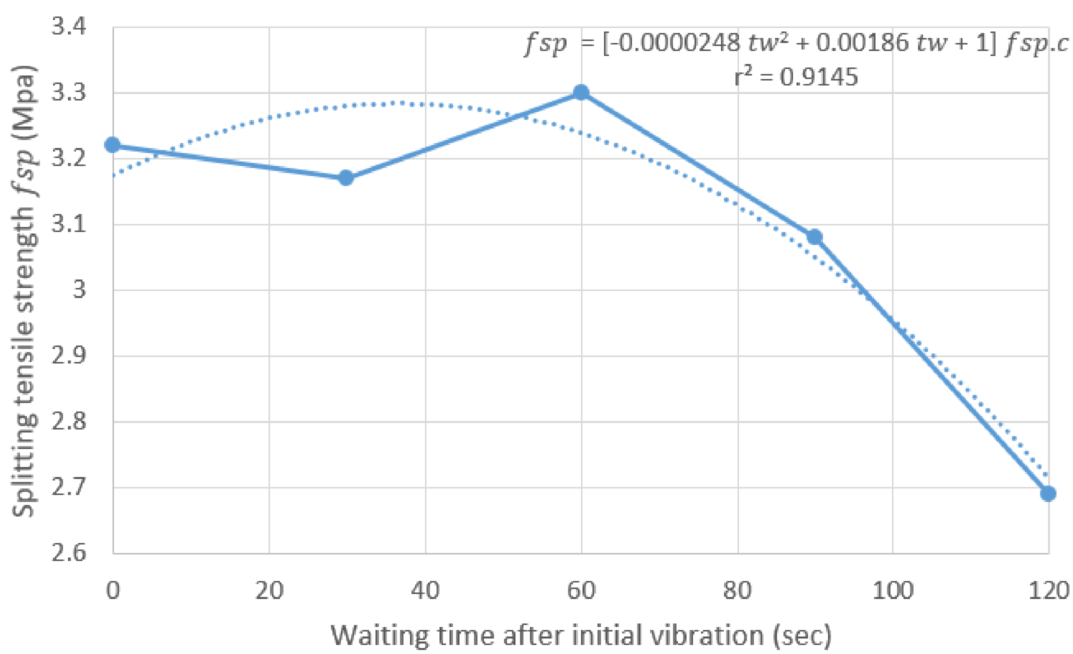

Fig. 4-10 Split tensile strengths of concrete with re-vibration time (60 sec) with different waiting time

4.3 Results and behavior of tested beams

The twenty-eight specimens were classified into seven groups (A, B, C, D, E, F and G) Table 4-11 shows the results of various beam specimens, and we can examine and compare the data to identify trends and variations.

The specimens are labeled from A1 to G4, and each row contains information about the initial vibration time, waiting time after initial vibration, re-vibration time, compressive strength ($f_c^{'}$), split tensile strength (f_{sp}), first crack load (P_{cr}), and ultimate load (P_u).

There are variations in compressive strength, splitting tensile strength, first crack load, and ultimate load among the different specimens. Factors such as initial vibration time, waiting time after initial vibration, and re-vibration time contribute to these variations. Further analysis and comparison can help identify specific relationships and trends within the data.

Table 4-11 First crack and ultimate load results of the beam

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength $f_c^{\prime}$ (MPa)	Splitting tensile strength f_{sp} (MPa)	First crack load P_{cr} (kN)	$\frac{P_{cr}}{P_{cr,o}}$	Ultimate load p_u (kN)	$\frac{p_u}{p_{u,o}}$	$\frac{p_u}{p_{cr}}$
1	A1	15	0	0	32.873	3.22	42	0.8	102	1.025	2.43
2	A2	30	0	0	34.2	3.5	44	0.82	104	1.041	2.37
3	A3	45	0	0	35.836	3.62	45	0.82	109	1.086	2.43
4	A4	60	0	0	30.953	3.05	47	0.921	100	1.011	2.13
5	B1	15	30	15	35.556	3.43	45	0.823	104	1.037	2.32
6	B2	15	30	30	37.213	3.58	47	0.84	113	1.122	2.40
7	B3	15	30	45	38.313	3.62	49	0.863	116	1.149	2.37
8	B4	15	30	60	33.75	3.17	46	0.864	107	1.072	2.32
9	C1	15	60	15	38.98	3.72	49	0.856	107	1.058	2.18
10	C2	15	60	30	39.766	3.85	51	0.882	114	1.125	2.23
11	C3	15	60	45	43.256	4.34	55	0.912	119	1.167	2.16
12	C4	15	60	60	38.2	3.3	53	0.935	111	1.100	2.09
13	D1	15	90	15	36.036	3.16	46	0.836	102	1.016	2.21
14	D2	15	90	30	38.013	3.31	48	0.849	109	1.080	2.27

15	D3	15	90	45	38.886	3.58	50	0.874	112	1.108	2.24
16	D4	15	90	60	33.93	3.08	47	0.88	107	1.072	2.27
17	E1	15	120	15	31.836	3.07	46	0.889	102	1.028	2.21
18	E2	15	120	30	33.703	3.22	48	0.902	107	1.072	2.23
19	E3	15	120	45	30.986	3.43	49	0.96	109	1.102	2.22
20	E4	15	120	60	30.793	2.69	43	0.845	105	1.062	2.44
21	F1	15	60	15	38.98	3.72	41		73.8		1.8
22	F2	15	60	30	39.766	3.85	43		78		1.82
23	F3	15	60	45	43.256	4.34	47		80		1.71
24	F4	15	60	60	38.2	3.3	42		70		1.67
25	GI	15	60	15	38.98	3.72			11.9		-
26	G2	15	60	30	39.766	3.85			14.5		-
27	G3	15	60	45	43.256	4.34			18.7		-
28	G4	15	60	60	38.2	3.3			9.5		-

Figure 4-11 focuses on comparing the results based on the first crack load (P_{cr}) values and re-vibration time length for different groups. In Group A (Specimens A1 to A4), the first crack load P_{cr} values gradually increase from 42 KN to 47 KN. Group B (Specimens B1 to B4) shows slight variations in the first crack load P_{cr} values, ranging from 45 KN to 49 KN. Group C (Specimens C1 to C4) exhibits a continuous increase in first crack load P_{cr}, with values ranging from 53 KN to 55 KN. Group D (Specimens D1 to D4) demonstrates fluctuating but relatively close first crack load P_{cr} values, ranging from 46 KN to 50 KN. Group E (Specimens E1 to E4) displays some variation in the first crack load P_{cr} values, with the highest value of 48 KN and the lowest value of 43 KN. Group F (Specimens F1 to F4) shows fluctuations in the first crack load P_{cr} values, with the highest value of 47 KN and the lowest values of 41 KN and 42 KN. Group G (Specimens G1 to G4) has significantly lower first crack load P_{cr} values, ranging from 9.5 KN to 18.7 KN, compared to the other groups. Overall, the groups exhibit different trends and variations in the first crack load P_{cr} values, with some groups showing consistent increases, while others have fluctuations or lower values.

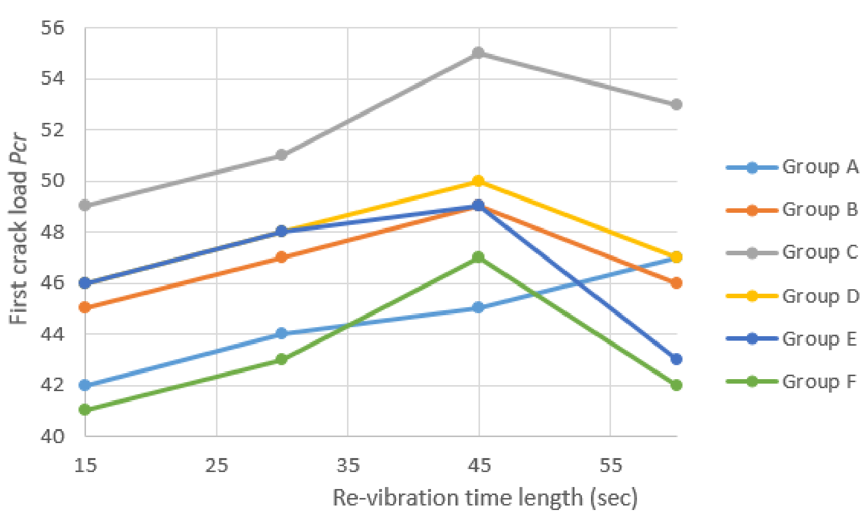

Fig. 4-2 First crack and Re-vibration time length results of the beam

Figure 4-12 shows the groups in the were analyzed based on several parameters. All groups had the same initial vibration time of 15 seconds. However, the waiting time after the initial vibration varied across the groups. Group A had no waiting time, Group B waited for 30 minutes, Group C waited for 60 minutes, Group D waited for 90 minutes, Group E waited for 120 minutes, and Groups F and G both waited for 60 min.

Re-vibration time was present in Groups B, C, D, E, F, and G, with varying durations from 15 to 60 seconds. Group A did not have any re-vibration time. The ultimate load P_u values showed variations among the groups. Group C had the highest ultimate load P_u values, ranging from 114 KN to 119 KN. Group F, which did not have stirrups, had ultimate load P_u values ranging from 70 KN to 80 KN. On the other hand, Group G, which is without reinforcement, had the lowest ultimate load P_u values overall, ranging from 9.5 KN to 18.7 KN. Overall, the analysis revealed that the groups had different waiting times after the initial vibration, and the ultimate load P_u values exhibited variations within and across the groups. Group C stood out with the highest ultimate load P_u values.

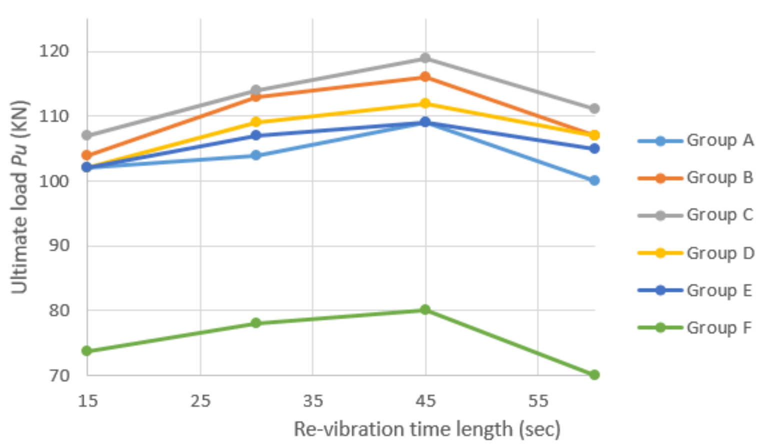

Fig. 4-3 Ultimate load and Re-vibration time length results of the beam

Table 4-12 and figure 4-13 consists of five specimens, A1 to E1, and their respective values for initial vibration time, waiting time after initial vibration, re-vibration time, and first crack load P_{cr}.

Specimen A1 serves as the reference point, with an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and a first crack load P_{cr} of 42 KN. Specimen B1 has the same initial vibration time as A1 but has a waiting time of 30 minutes after the initial vibration. It also has a re-vibration time of 15 seconds and a slightly higher first crack load P_{cr} of 45 KN.

Specimen C1 has the same initial vibration time and re-vibration time as A1, but its waiting time after the initial vibration is extended to 60 minutes. Its first crack load P_{cr} further increases to 49 KN. Specimen D1 shares the same initial vibration time, re-vibration time, and waiting time after the initial vibration as A1, but its first crack load P_{cr} is slightly lower at 46 KN.

Lastly, Specimen E1 has the same initial vibration time, re-vibration time, and waiting time after the initial vibration as A1. Its first crack load P_{cr} is also 46 KN, identical to that of D1.

Comparing these specimens, we can observe that the waiting time after the initial vibration has an impact on the first crack load P_{cr}. As the waiting time increases from B1 to C1, the first crack load P_{cr} also increases. However, there is no significant difference in the first crack load P_{cr} between D1, E1, and A1, indicating that the waiting time beyond 60 minutes does not contribute to a significant change in the first crack load P_{cr}.

Table 4-12 First crack and ultimate load of the beam with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f_c' (MPa)	Splitting tensile strength f_{sp} (MPa)	First crack load P_{cr} (KN)	$\frac{P_{cr}}{P_{cr,o}}$	Ultimate load P_u (KN)	$\frac{P_u}{P_{u,o}}$	$\frac{P_u}{P_{cr}}$
1	A1	15	0	0	32.873	3.22	42	0.8	102	1.025	2.43
2	B1	15	30	15	35.556	3.43	45	0.823	104	1.037	2.32
3	C1	15	60	15	38.98	3.72	49	0.856	107	1.058	2.18
4	D1	15	90	15	36.036	3.16	46	0.836	102	1.016	2.21
5	E1	15	120	15	31.836	3.07	46	0.889	102	1.028	2.21

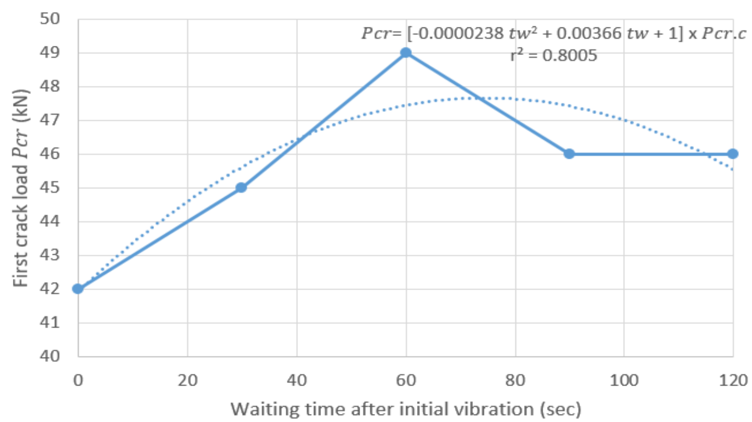

Fig. 4-13 First crack results of the beam with re-vibration time (15 sec) with different waiting time

Table 4-12 and figure 4-14 shows five specimens (A1 to E1) are examined based on their initial vibration time, waiting time after initial vibration, re-vibration time, and ultimate load P_u. The results show that the waiting time and re-vibration time have minimal influence on the ultimate load P_u. Specimens with different waiting times and re-vibration times demonstrate similar ultimate load P_u values. Specimen C1 has a slightly higher ultimate load P_u of 107 KN compared to the other specimens, which range from 102 KN to 104 KN. These findings suggest that factors other than waiting and re-vibration times may play a more significant role in determining the ultimate load P_u. Further analysis and additional data are required to gain a deeper understanding of the relationships between variables and the ultimate loadP_u.

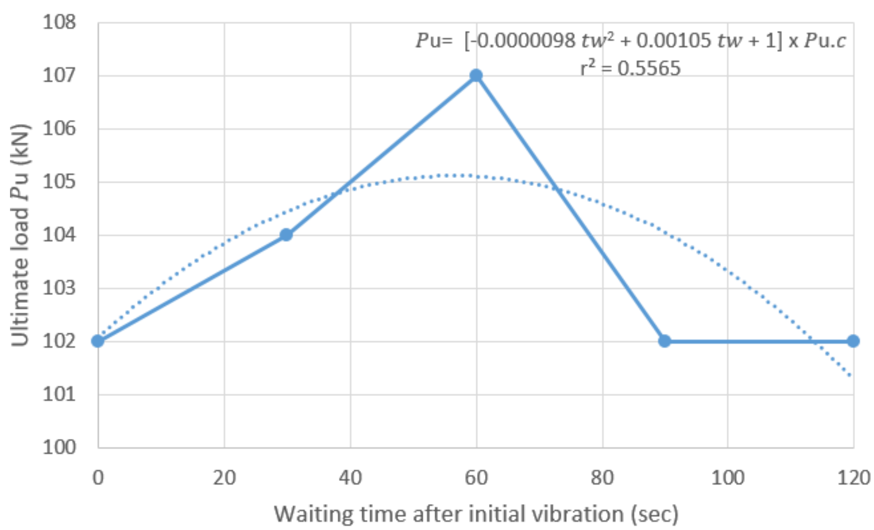

Fig. 4-4 Ultimate load results of the beam with re-vibration time (15 sec)with different waiting time

Table 4-13 and figure 4-15 presents data for five specimens (A1 to E2) including their initial vibration time, waiting time after initial vibration, re-vibration time, and first crack load P_{cr}. Increasing the waiting time after the initial vibration from 30 to 60 minutes leads to a gradual increase in the first crack load P_{cr}. Specimen C2 with a waiting time of 60 minutes exhibits the highest first crack load P_{cr} of 51 KN. However, there is no significant difference in the first crack load P_{cr} between specimens D2, E2, and B2, suggesting that waiting times beyond 90 minutes do not significantly affect the first crack load P_{cr}. The re-vibration time of 30 seconds shows no notable impact on the first crack load P_{cr} across specimens B2, C2, D2, and E2.

Table 4-13 First crack and ultimate load of the beam with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f_c' (MPa)	Splitting tensile strength f_{sp} (MPa)	First crack load P_{cr} (kN)	$\frac{P_{cr}}{P_{cr,o}}$	Ultimate load P_u (kN)	$\frac{P_u}{P_{u,o}}$	$\frac{P_u}{P_{cr}}$
1	A1	15	0	0	32.873	3.22	42	0.8	102	1.025	2.43
2	B2	15	30	30	37.213	3.58	47	0.84	113	1.122	2.40
3	C2	15	60	30	39.766	3.85	51	0.882	114	1.125	2.23
4	D2	15	90	30	38.013	3.31	48	0.849	109	1.080	2.27
5	E2	15	120	30	33.703	3.22	48	0.902	107	1.072	2.23

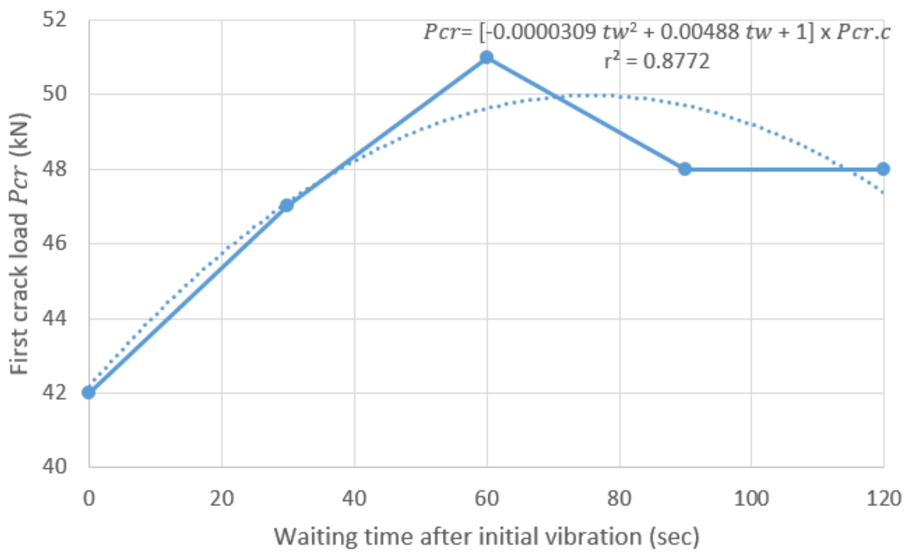

Fig. 4-5 First crack results of beam with re-vibration time (30 sec) with different waiting time

Table 4-13 and figure 4-16 includes five specimens (A1 to E2) with their respective values for initial vibration time, waiting time after initial vibration, re-vibration time, and ultimate load P_u.

Increasing the waiting time after initial vibration generally leads to higher ultimate load P_u values. Specimen C2, with a waiting time of 60 minutes, shows the highest ultimate load P_u (114 KN) among the specimens. Specimens B2 and D2 also have higher ultimate load P_u values compared to the reference point, A1.

However, Specimen E2, with a waiting time of 120 minutes, has a slightly lower ultimate load P_u of 107 KN. The re-vibration time of 30 seconds does not significantly affect the ultimate load P_u, as specimens with the same re-vibration time exhibit similar values.

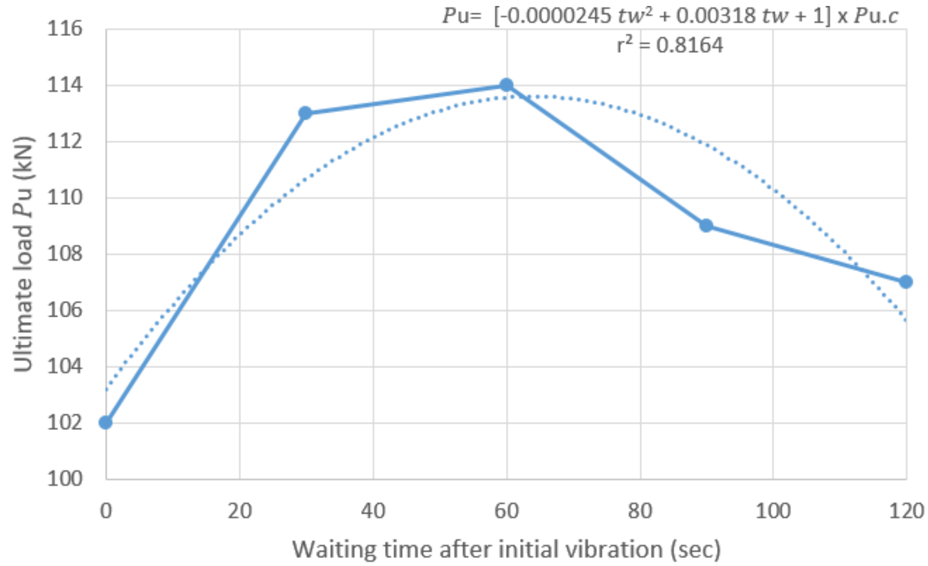

Fig. 4-6 Ultimate load of beam with re-vibration time (30 sec) with different waiting time

Table 4-14 and figure 4-17 presents data on specimens (A1 to E3) and their characteristics, including initial vibration time, waiting time after initial vibration, re-vibration time, and first crack load P_{cr}.

Specimen A1 serves as the reference with an initial vibration time of 15 seconds, no waiting time or re-vibration time, and a first crack load P_{cr} of 42 KN.

As the waiting time after initial vibration increases, specimens B3, C3, and E3 show higher first crack load P_{cr} values, reaching 49 KN, 55 KN, and 49 KN, respectively. However, specimen D3 deviates from this trend with a slightly lower first crack load P_{cr} of 50 KN despite the longer waiting time of 90 minutes.

The re-vibration time of 45 seconds appears to have a minimal impact on the first crack load P_{cr}, as specimens B3, C3, D3, and E3 with the same re-vibration time show similar first crack load P_{cr} values.

In summary, increasing the waiting time up to 60 minutes generally leads to higher first crack load P_{cr} values. However, waiting times beyond 90 minutes do not necessarily result in further increases. The re-vibration time of 45 seconds does not significantly affect the first crack load P_{cr}. Further analysis is needed to confirm these trends and draw more conclusive insights.

Table 4-14 First crack and ultimate load of the beam with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength f_c' (MPa)	Splitting tensile strength f_{sp} (MPa)	First crack load P_{cr} (kN)	$\frac{P_{cr}}{P_{cr,o}}$	Ultimate load P_u (kN)	$\frac{P_u}{P_{u,o}}$	$\frac{P_u}{P_{cr}}$
1	A1	15	0	0	32.873	3.22	42	0.8	102	1.025	2.43
2	B3	15	30	45	38.313	3.62	49	0.863	116	1.149	2.37
3	C3	15	60	45	43.256	4.34	55	0.912	119	1.167	2.16
4	D3	15	90	45	38.886	3.58	50	0.874	112	1.108	2.24
5	E3	15	120	45	30.986	3.43	49	0.96	109	1.102	2.22

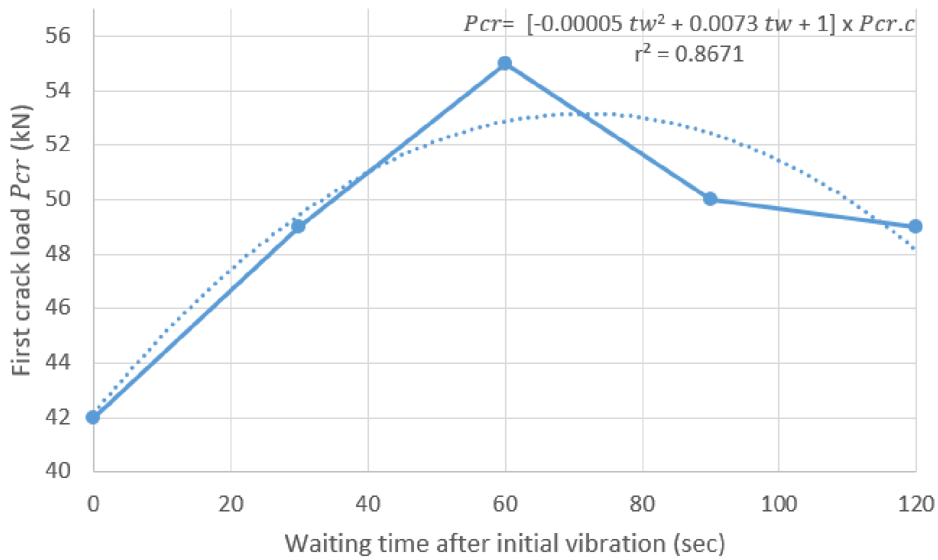

Fig. 4-7 First crack results of beam with re-vibration time (45 sec) with different waiting time

Table 4-14 and figure 4-18 presents data on specimens (A1 to E3) and their characteristics, including initial vibration time, waiting time after initial vibration, re-vibration time, and ultimate load P_u.

Increasing the waiting time after initial vibration up to 60 minutes leads to higher ultimate load P_u values in specimens B3, C3, D3, and E3, with values reaching 116 KN, 119 KN, 112 KN, and 109 KN, respectively. This indicates improved structural performance with longer waiting times. For re-vibration time of 45 seconds the ultimate load P_u increases for waiting time 30 and 60 minutes and then decreases.

In summary, increasing the waiting time up to 60 minutes enhances the ultimate load P_u, suggesting improved structural strength. However, further analysis is needed to confirm these findings and draw more comprehensive conclusions.

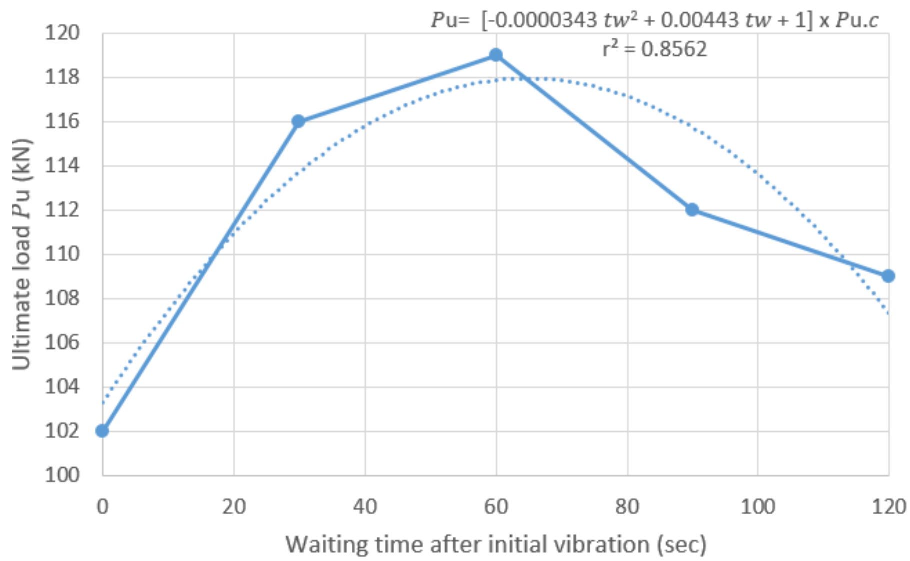

Fig. 4-8 Ultimate load of beam with re-vibration time (45 sec) with different waiting time

Table 4-15 and figure 4-19 presents data for specimens (A1 to E4) regarding their initial vibration time, waiting time after initial vibration, re-vibration time, and first crack load P_{cr}. Upon examining the data, it becomes apparent that increasing the waiting time after the initial vibration does not consistently lead to higher first crack load P_{cr} values. Specimen C4, which has the longest waiting time of 60 minutes, displays the highest first crack load P_{cr} value of 53 KN. In contrast, Specimen E4, with a waiting time of 120 minutes, exhibits the lowest first crack load P_{cr} value of 43 KN. This suggests that other factors beyond the waiting time may influence the first crack load P_{cr}.

Furthermore, the re-vibration time of 60 seconds does not seem to significantly impact the first crack load P_{cr}. Specimens B4, C4, D4, and E4, all sharing the same re-vibration time, showcase varying first crack load P_{cr} values.

Table 4-15 First crack and ultimate load of the beam with different waiting time

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Compressive strength $f_c^{'}$ (MPa)	Splitting tensile strength f_{sp} (MPa)	First crack load P_{cr} (kN)	$\frac{P_{cr}}{P_{cr},o}$	Ultimate load P_u (kN)	$\frac{P_u}{P_u,o}$	$\frac{P_u}{P_{cr}}$
1	A1	15	0	0	32.873	3.22	42	0.8	102	1.025	2.43
2	B4	15	30	60	33.75	3.17	46	0.864	107	1.072	2.32
3	C4	15	60	60	38.2	3.3	53	0.935	111	1.100	2.09
4	D4	15	90	60	33.93	3.08	47	0.88	107	1.072	2.27
5	E4	15	120	60	30.793	2.69	43	0.845	105	1.062	2.44

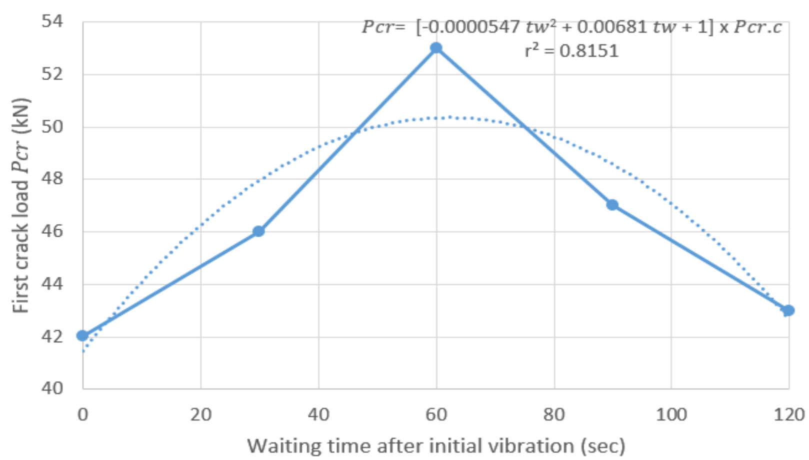

Fig. 4-9 First crack results of beam with re-vibration time (60 sec) with different waiting time

Table 4-15 and figure 4-20 contains data for specimens (A1 to E4) with their respective values for initial vibration time, waiting time after initial vibration, re-vibration time, and ultimate load P_u.

Specimen A1 serves as the reference point with an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and an ultimate load P_u of 102 KN.

Increasing the waiting time after the initial vibration results in varying ultimate load P_u values for specimens B4, C4, D4, and E4. Specimen B4, with a 30-minute waiting time and a re-vibration time of 60 seconds, has an ultimate load P_u of 107 KN. Specimen C4, with a 60-minute waiting time and the same re-vibration time, exhibits a higher ultimate load P_u of 111 KN. Specimen D4, with a waiting time of 90 minutes, shares the same ultimate load P_u as B4 at 107 KN. Specimen E4, with a waiting time of 120 minutes, has a slightly lower ultimate load P_u of 105 KN.

Comparing the specimens, it is evident that increasing the waiting time after the initial vibration can impact the ultimate load P_u, but other factors may also influence the variations observed. Specimen C4, with the longest waiting time of 60 minutes, shows the highest ultimate load P_u at 111 KN. Specimen E4, with the longest waiting time of 120 minutes, has a slightly lower ultimate load P_u. However, specimens B4 and D4, despite having different waiting times, share the same ultimate load P_u value of 107 KN.

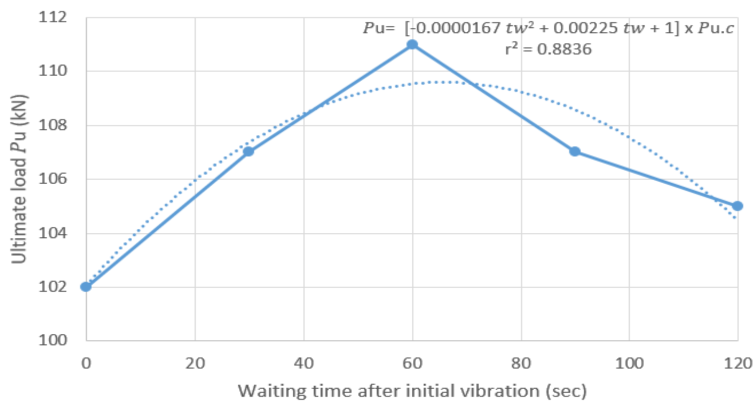

Fig. 4-10 Ultimate load of beam with re-vibration time (60 sec) with different waiting time

4.4 Crack Pattern and Mode of Failure

Several crack patterns and failure modes were observed in the experiments. The failure modes of the reinforced concrete beams under variant re-vibration times and waiting time of the vibration the beams were divided into seven groups each group consists of four samples. The beams for group (A, B, C, D, E and F) were reinforced beams, and group F which are without transverse reinforcement and group G is without longitudinal and transverse reinforcement. In group (A, B, C, D and E) were the beams subjected to applied load during the tests, initial cracks were appeared hairline cracks and distributed spirally across the section along with an amount of small inclined cracks. The first crack appears at the center of the beams. These cracks dispersed throughout both sides of the beams as shown in figures 4-21 till 4-25. In group F, beam specimens (F1, F2, F3 and F4), the first crack occurred near the ends of the lever arm on both sides of the specimens, it was found that the cracks developed an angle of (45^o) with the sides of the beams as shown in figure 4-26. In group G, beam specimens (G1, G2, G3, and G4) sudden failure of the beams appeared after the appearance of the initial cracks as shown in figure 4-27.

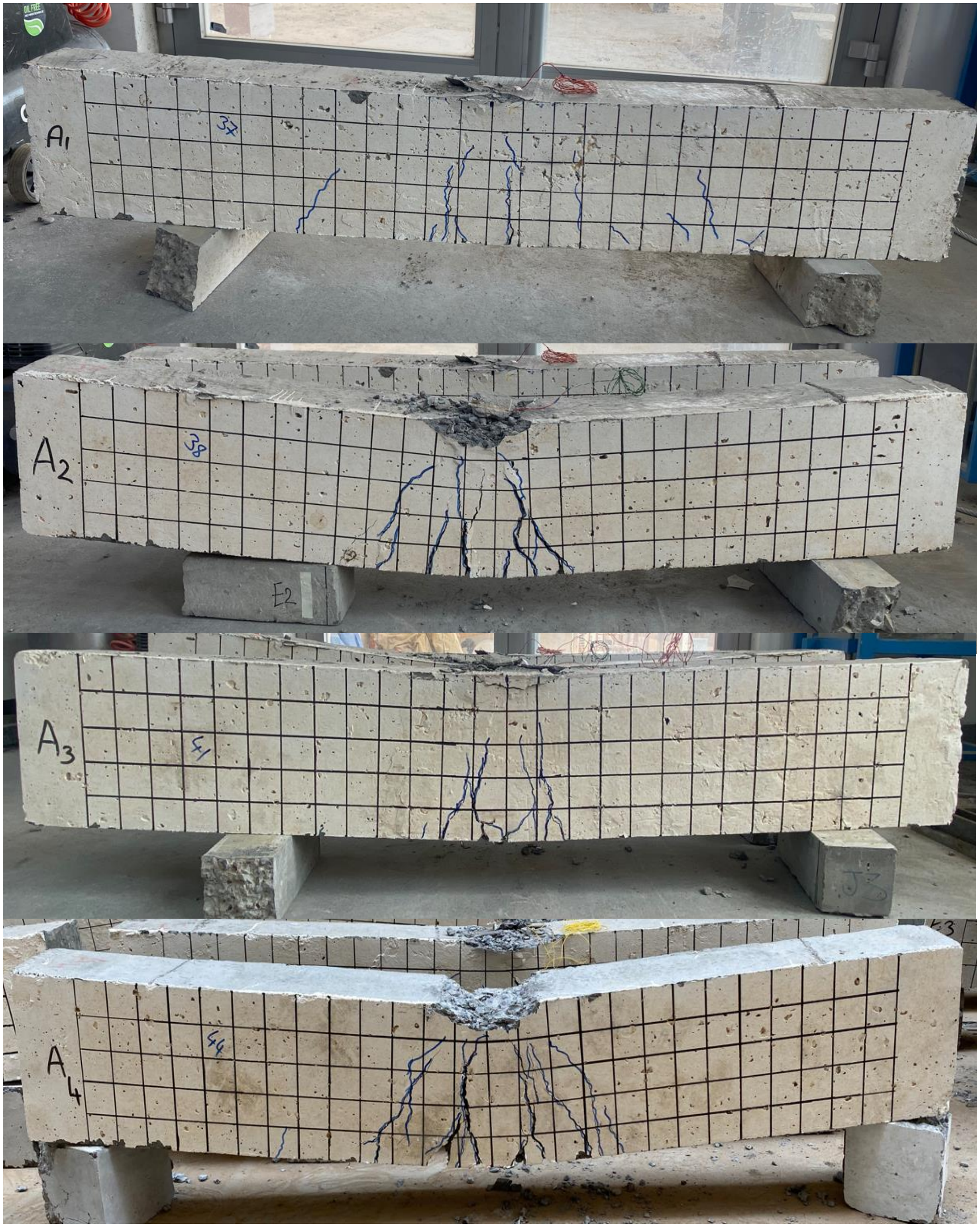

Fig. 4-21 Group (A) Cracking pattern of the test specimens (A1, A2, A3 and A4)

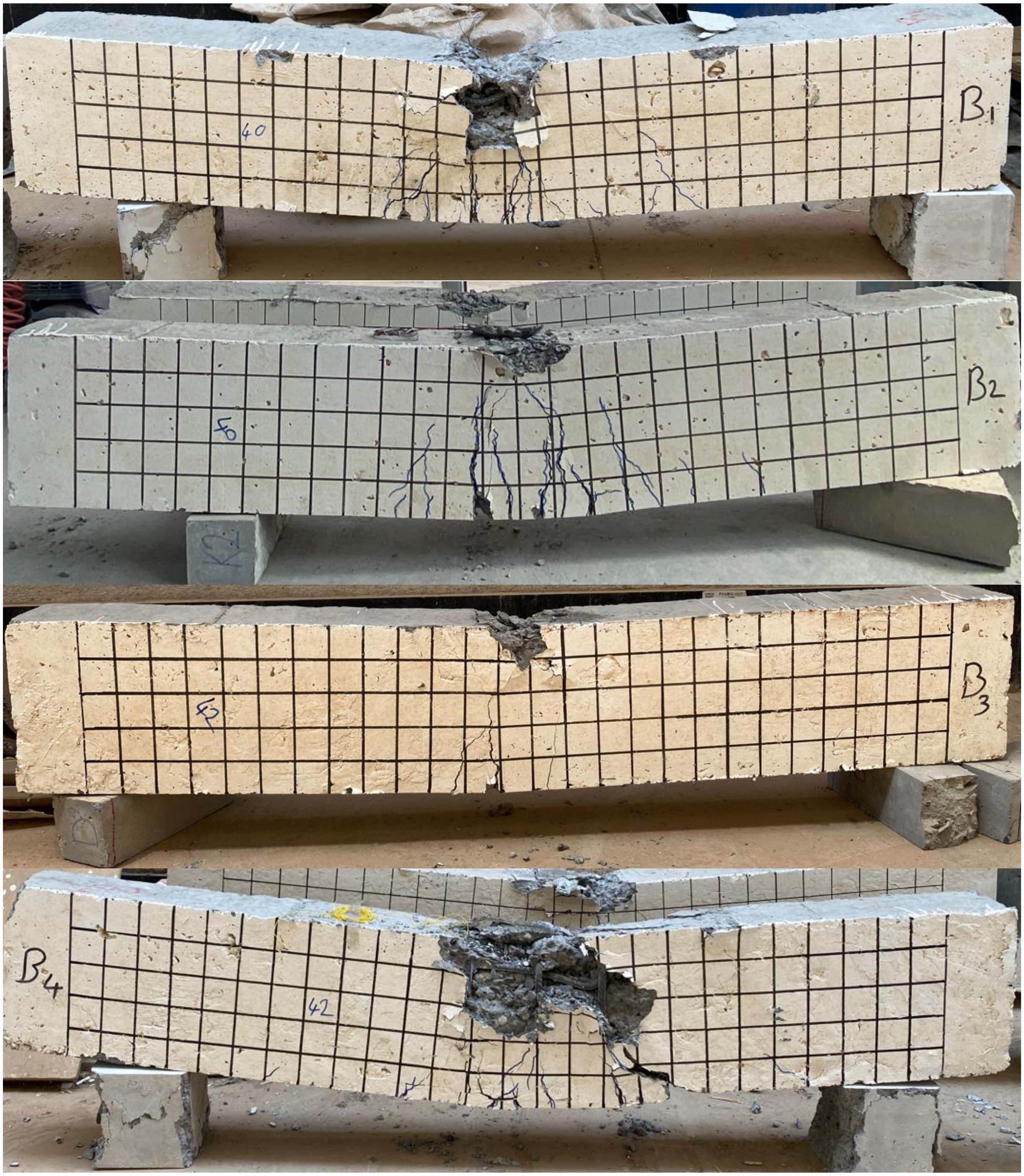

Fig. 4-11 Group (B) Cracking pattern of the test specimens (B1, B2, B3 and B4)

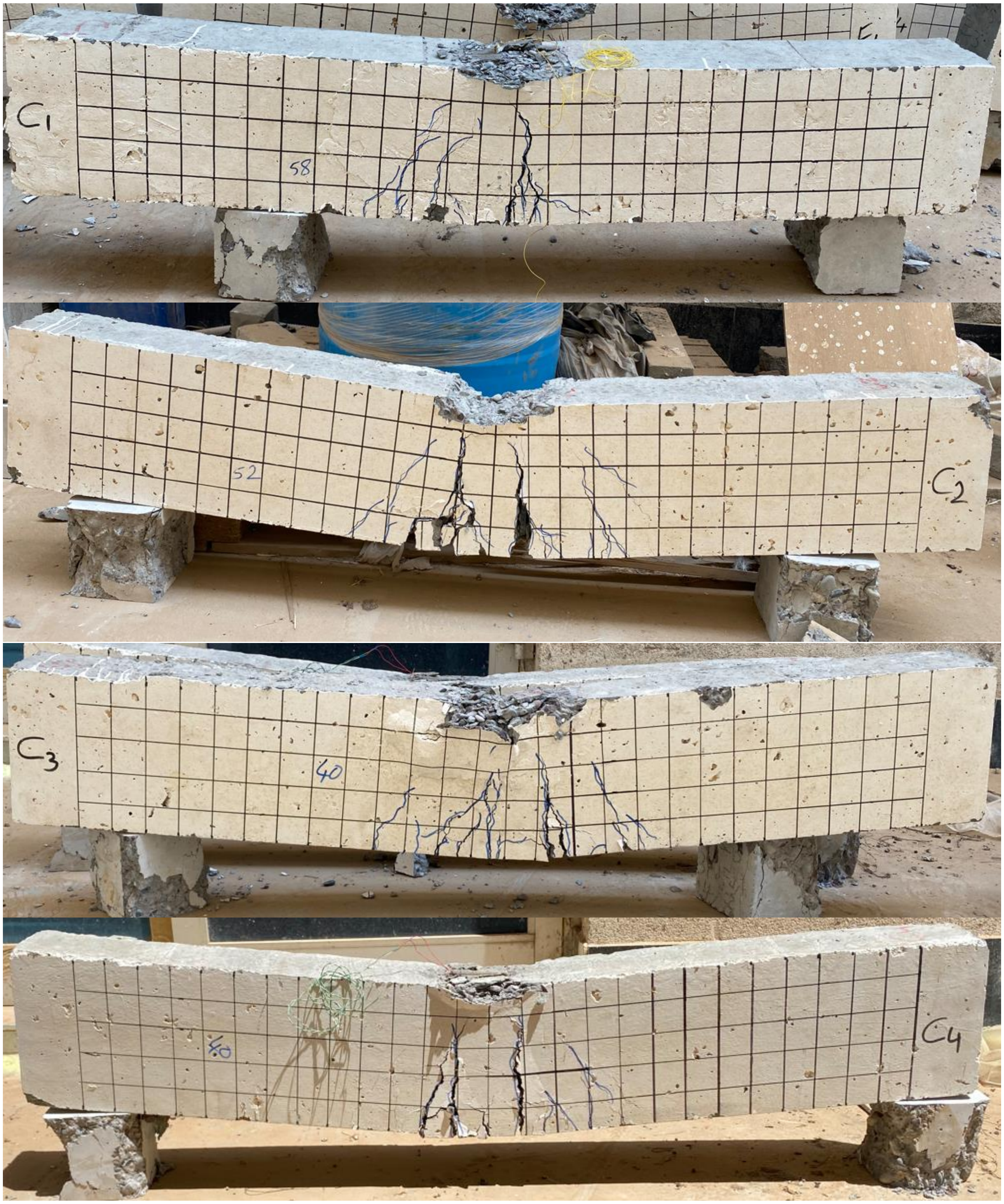

Fig. 4-12 Group (C) Cracking pattern of the test specimens (C1, C2, C3 and C4)

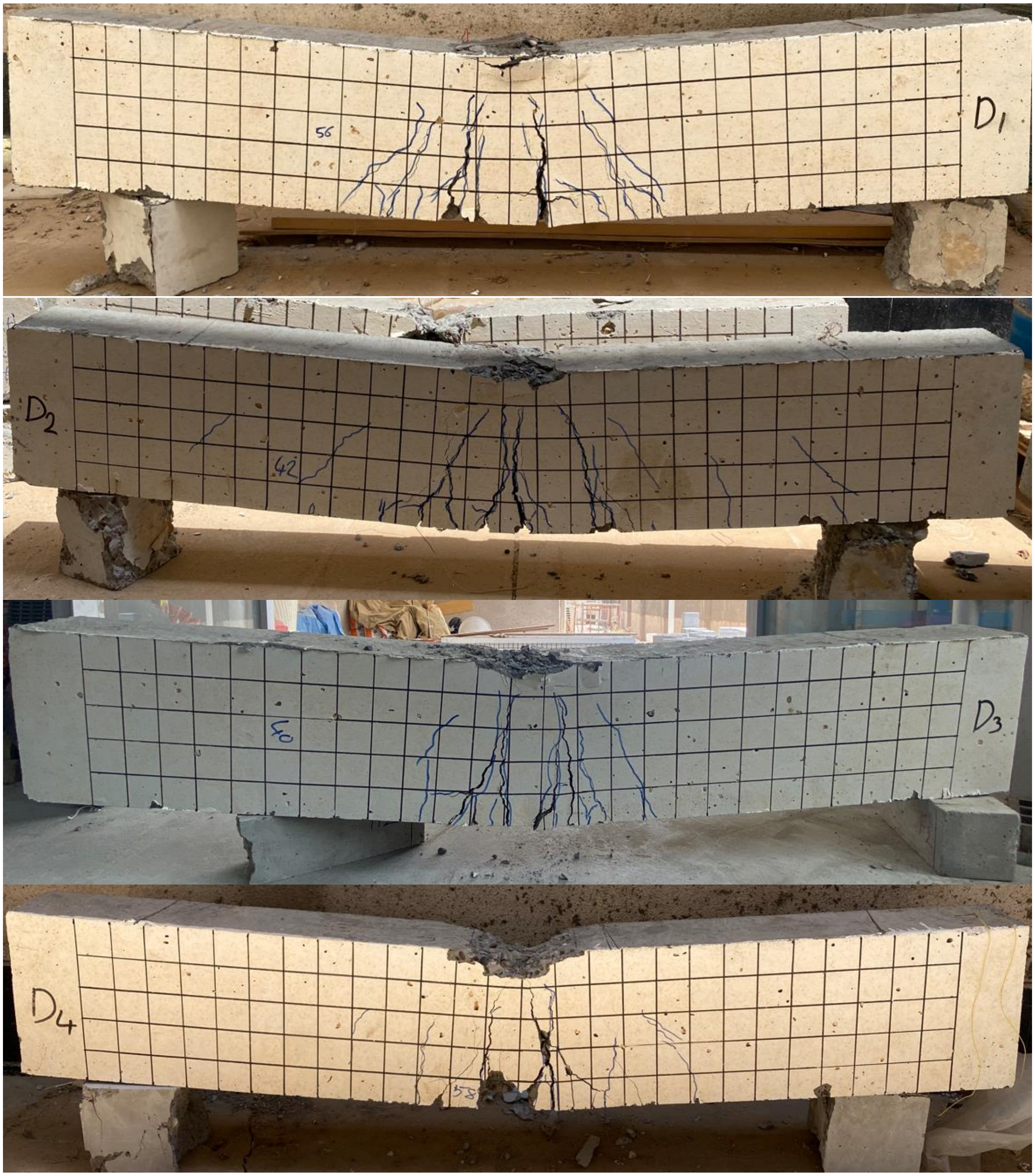

Fig. 4-13 Group (D) Cracking pattern of the test specimens (D1, D2, D3 and D4)

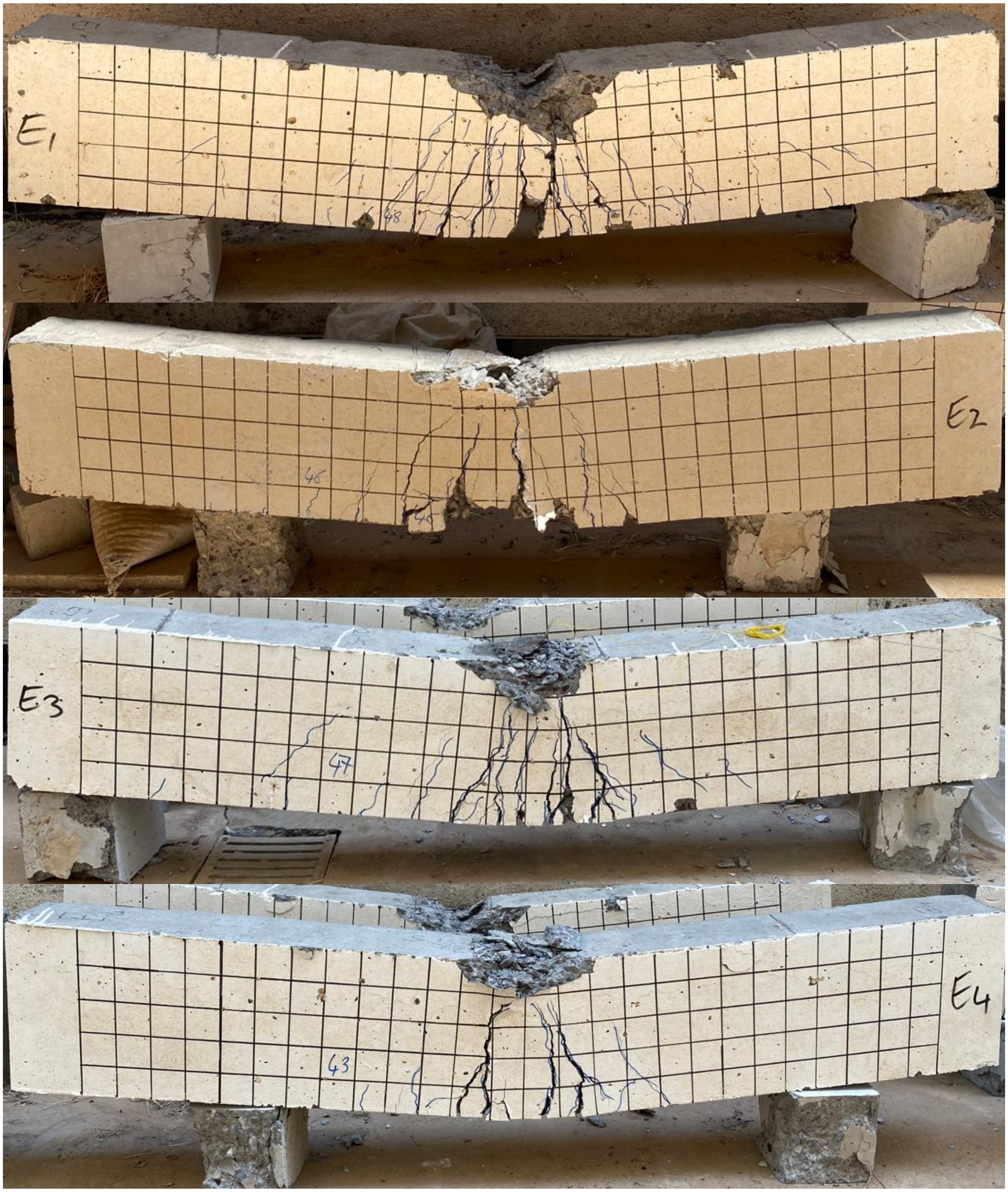

Fig. 4-14 Group (E) Cracking pattern of the test specimens (E1, E2, E3 and E4)

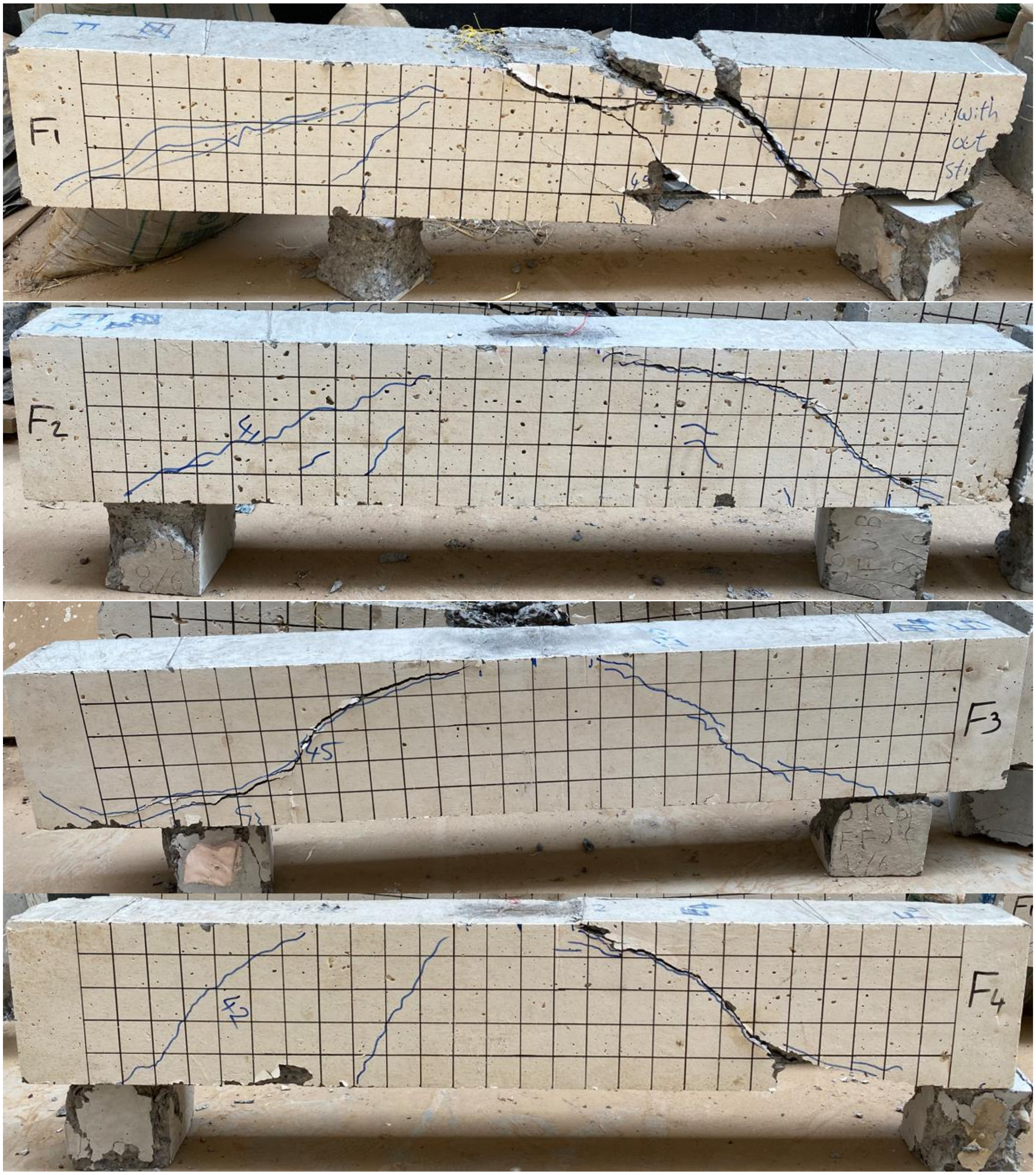

Fig. 4-15 Group (F) Cracking pattern of the test specimens (F1, F2, F3 and F4)

Fig. 4-16 Group (G) Cracking pattern of the test specimens (G1, G2, G3 and G4)

4.5 Measured Load-Deflection Curves

All the beams when subjected to the load, at the beginning any section of the beams will behave linearly, the concrete at the tension zone will arrive its tensile strength at that moment the cracks in tension zone began to appear (like hair cracks). by little increasing the load the number and width of the crack will be increase.

During this procedure also the strain of concrete in compression zone is recorded with the value of deflection of the beams at the mid span. Table 4-16 show the strain, deflection, first crack load and finally the stiffness for all the beams.

Table 4-16 Strain and Deflection results of the beam

No.	Specimen	Initial Vibration time (sec)	Waiting time after initial vibration (min)	Re-vibration time (sec)	Strain	Deflection at ultimate load δu (mm)	First crack load Pcr (kN)	Deflection at cracking load δ_{cr} (mm)	Stiffness $k = \frac{P_{cr}}{\delta_{cr}}$
1	A1	15	0	0	0.001726	5.5	42	1.2	35
2	A2	30	0	0	0.002075	7.8	44	1.45	30.34
3	A3	45	0	0	0.002188	9	45	1.8	25
4	A4	60	0	0	0	6.9	47	2	23.5
5	B1	15	30	15	0	8.1	45	2.15	20.93
6	B2	15	30	30	0.001751	8.5	47	1.9	24.73
7	B3	15	30	45	0.002085	10.2	49	1.5	32.66
8	B4	15	30	60	0.001415	7.78	46	2.3	20
9	C1	15	60	15	0.001978	9.1	49	1.8	27.22
10	C2	15	60	30	0.002182	10.2	51	2.2	23.18

11	C3	15	60	45	0.002987	13.35	55	2.05	26.82
12	C4	15	60	60	0.001752	8.5	53	2.75	19.27
13	D1	15	90	15	0.001518	8	46	1.92	23.95
14	D2	15	90	30	0.001602	9.5	48	2.2	21.81
15	D3	15	90	45	0.001704	10.2	50	2.3	21.73
16	D4	15	90	60	0	7.8	47	2	23.5
17	E1	15	120	15	0.001681	7.3	46	2.05	22.43
18	E2	15	120	30	0.001819	8.1	48	1.9	25.26
19	E3	15	120	45	0.002073	8.5	49	2.3	21.3
20	E4	15	120	60	0	6.9	43	1.8	23.88
21	F1	15	60	15	0.00071	4	41	1.5	27.33
22	F2	15	60	30	0.000932	4.2	43	1.7	25.29
23	F3	15	60	45	0.001102	5.1	47	1.9	24.73
24	F4	15	60	60	0.000689	3.55	42	1.2	35
25	GI	15	60	15	0	0			
26	G2	15	60	30	0	0			
27	G3	15	60	45	0	0			
28	G4	15	60	60	7.29E-05	0			

Figure 4-28 till figure 4-33 shows that the ultimate load deflection at re-vibration time 45 sec. for all waiting time 30, 60, 90, and 120 min.

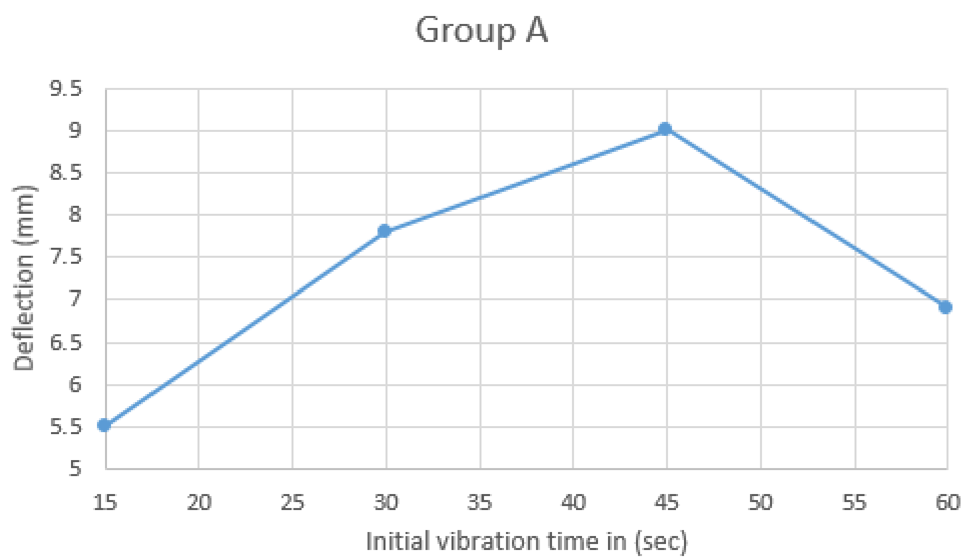

Fig. 4-28 Ultimate load and deflection curve of group A

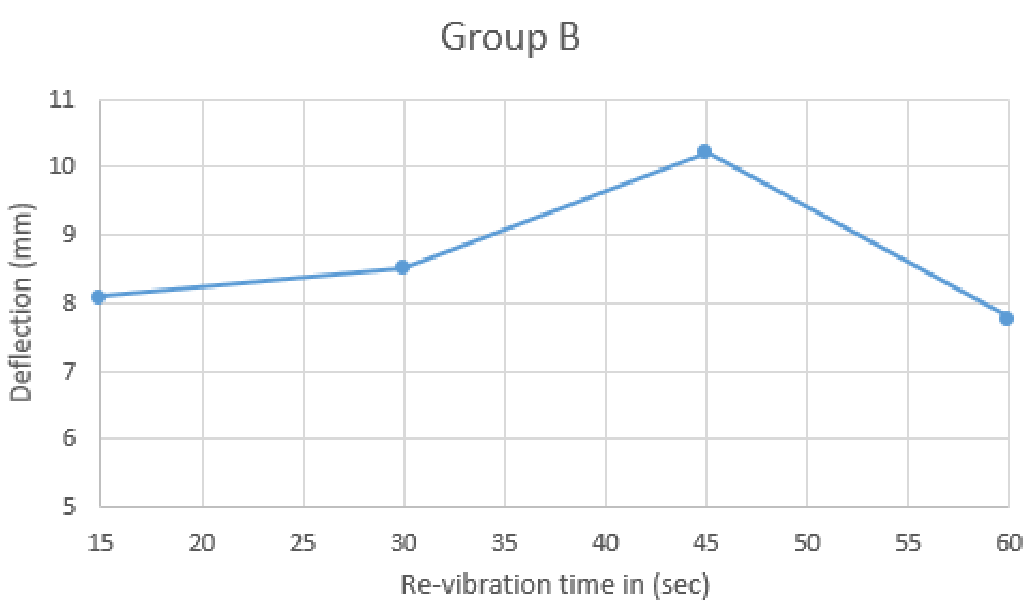

Fig. 4-29 Ultimate load and deflection curve of group B

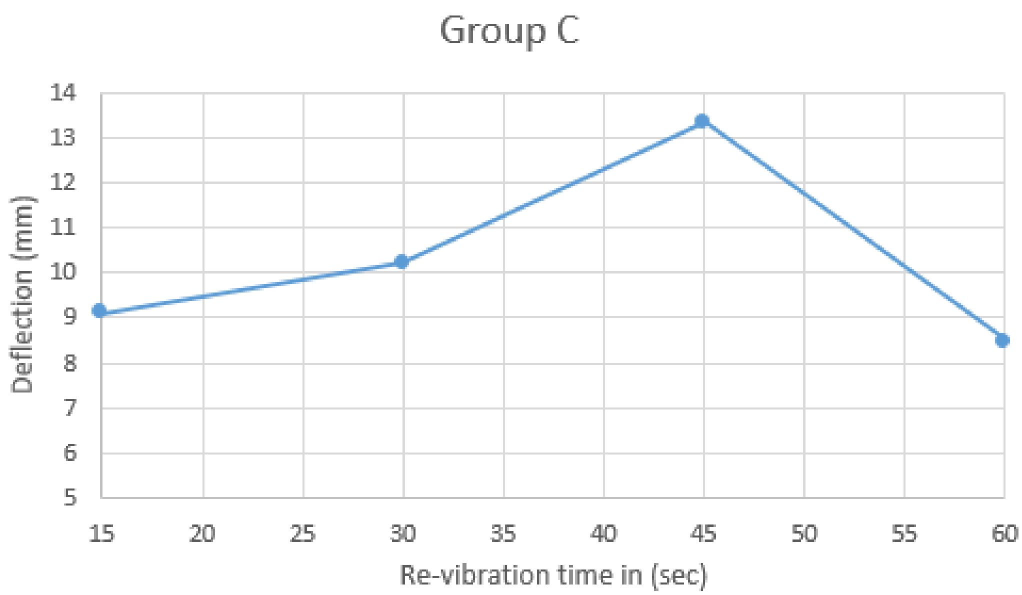

Fig. 4-30 Ultimate load and deflection curve of group C

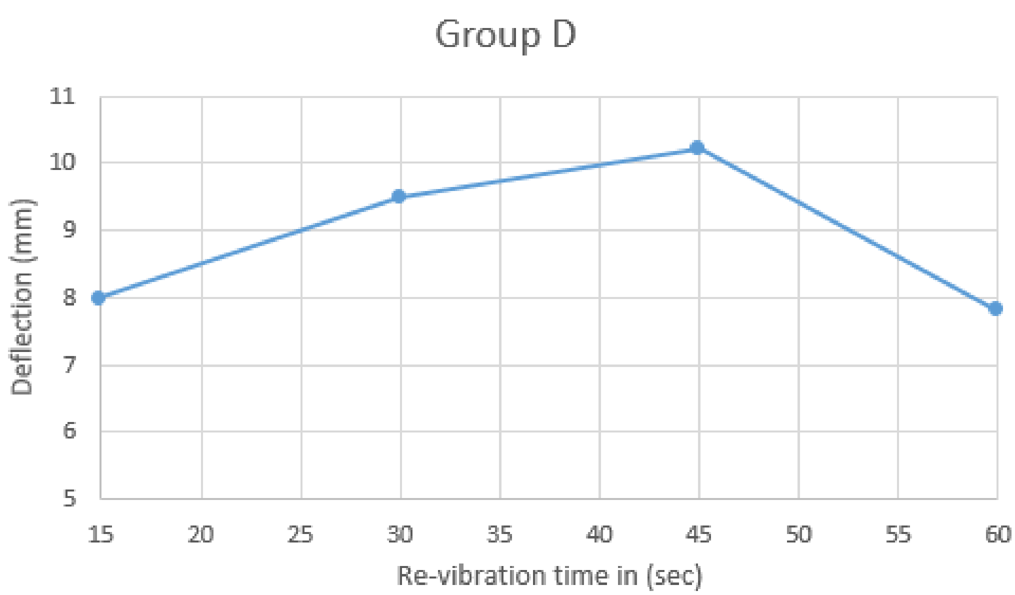

Fig. 4-31 Ultimate load and deflection curve of group D

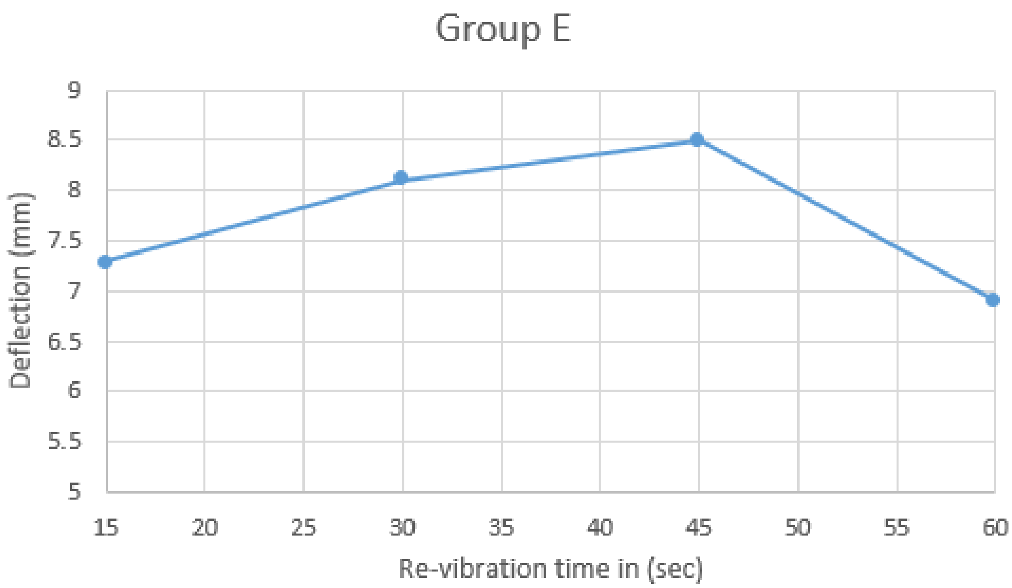

Fig. 4-32 Ultimate load and deflection curve of group E

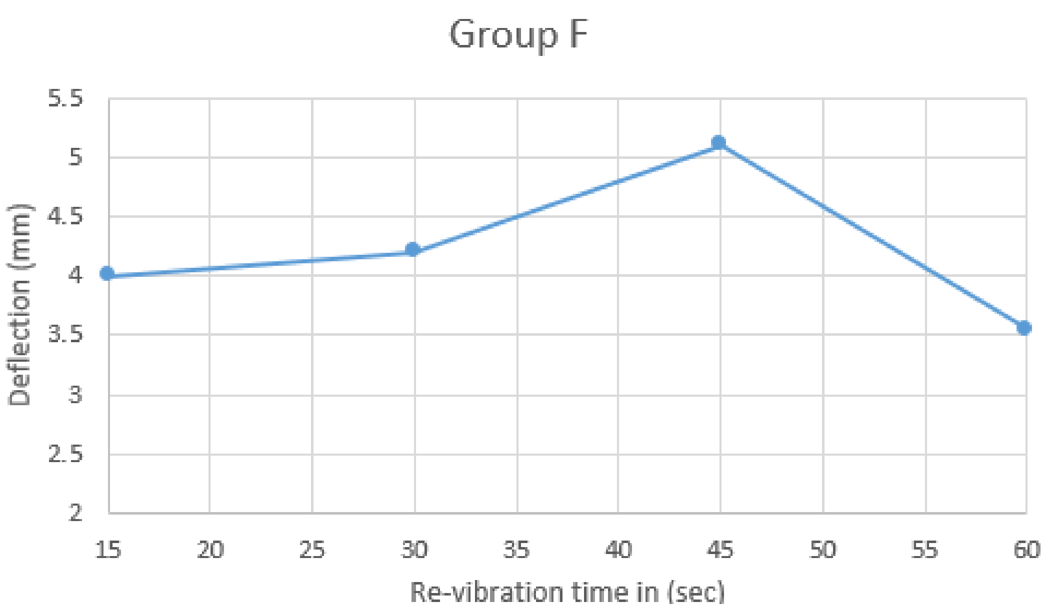

Fig. 4-33 Ultimate load and deflection curve of group F

Table 4-17 and figure 4-34 describes the characteristics of specimens (A1 to E1) in terms of their initial vibration time, waiting time after the initial vibration, re-vibration time, and deflection at the ultimate load.

Specimen A1 serves as the reference point, exhibiting an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and a deflection of 5.5 mm at the ultimate load.

Increasing the waiting time after the initial vibration leads to higher deflection values at the ultimate load for specimens B1, C1, D1, and E1. Specimen B1, with a waiting time of 30 minutes and a re-vibration time of 15 seconds, has a deflection of 8.1 mm. Specimen C1, with a waiting time of 60 minutes and the same re-vibration time, displays a further increase in deflection to 9.1 mm. Specimen D1, with a waiting time of 90 minutes, has a deflection of 8.0 mm, while Specimen E1, with a waiting time of 120 minutes, exhibits a deflection of 7.3 mm.

In summary, the data indicates that increasing the waiting time after the initial vibration generally leads to higher deflection values at the ultimate load.

Table 4-17 Strain and Deflection results of the beam for different waiting time

No.	Specimen	Initial Vibration time (sec)	w_t after initial vibration (min)	t_{rv} (sec)	Strain	δu (mm)	First crack load P_{cr} (kN)	δ_{cr} (mm)	Stiffness $k = \frac{P_{cr}}{\delta_{cr}}$
1	A1	15	0	0	0.001726	5.5	42	1.2	35
2	B1	15	30	15	0	8.1	45	2.15	20.93
3	C1	15	60	15	0.001978	9.1	49	1.8	27.22
4	D1	15	90	15	0.001518	8	46	1.92	23.95
5	E1	15	120	15	0.001681	7.3	46	2.05	22.43

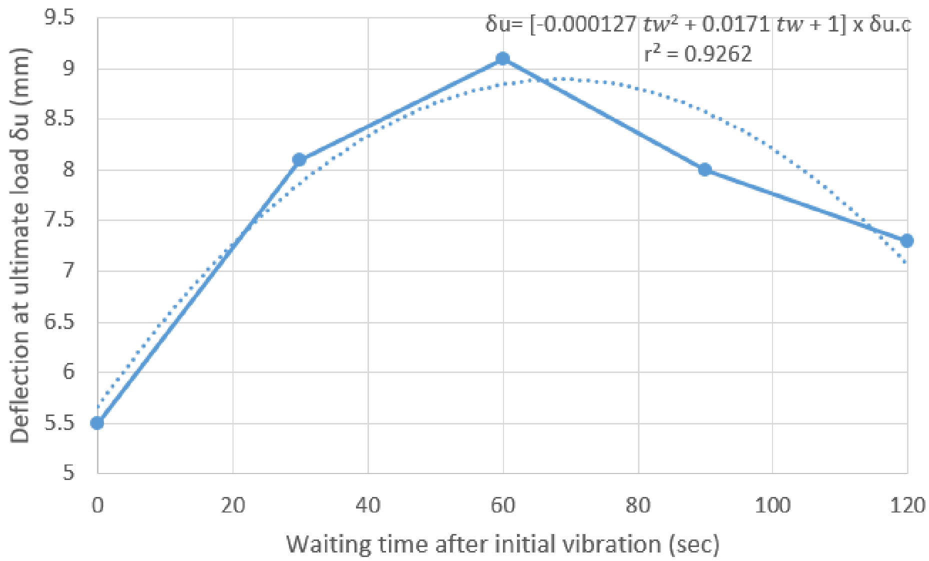

Fig. 4-34 Ultimate load deflection results of the beam with waiting time for re-vibration time length (15 sec.)

Table 4-18 and figure 4-35 compares the results of specimens (A1 to E2) in terms of their initial vibration time, waiting time after the initial vibration, re-vibration time, and deflection at the ultimate load. Specimen A1 serves as the reference point, with an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and a deflection of 5.5 mm at the ultimate load.As the waiting time after the initial vibration increases, specimens B2, C2, D2, and E2 demonstrate higher deflection values at the ultimate load. Specimen B2, with a waiting time of 30 minutes and a re-vibration time of 30 seconds, exhibits a deflection of 8.5 mm. Specimen C2, with a waiting time of 60 minutes and the same re-vibration time, shows a further increase in deflection to 10.2 mm. Specimen D2, with a waiting time of 90 minutes, has a deflection of 9.5 mm. Lastly, Specimen E2, with a waiting time of 120 minutes, displays a deflection of 8.1 mm. The results evident that increasing the waiting time after the initial vibration generally leads to higher deflection values at the ultimate load.

Table 4-18 Strain and Deflection results of the beam for different waiting time

No.	Specimen	Initial Vibration time (sec)	w_t after initial vibration (min)	t_{rv} (sec)	Strain	δu (mm)	P_{cr} (KN)	δ_{cr}	Stiffness $k = \frac{P_{cr}}{\delta_{cr}}$
1	A1	15	0	0	0.001726	5.5	42	1.2	35
2	B2	15	30	30	0.001751	8.5	47	1.9	24.73
3	C2	15	60	30	0.002182	10.2	51	2.2	23.18
4	D2	15	90	30	0.001602	9.5	48	2.2	21.81
5	E2	15	120	30	0.001819	8.1	48	1.9	25.26

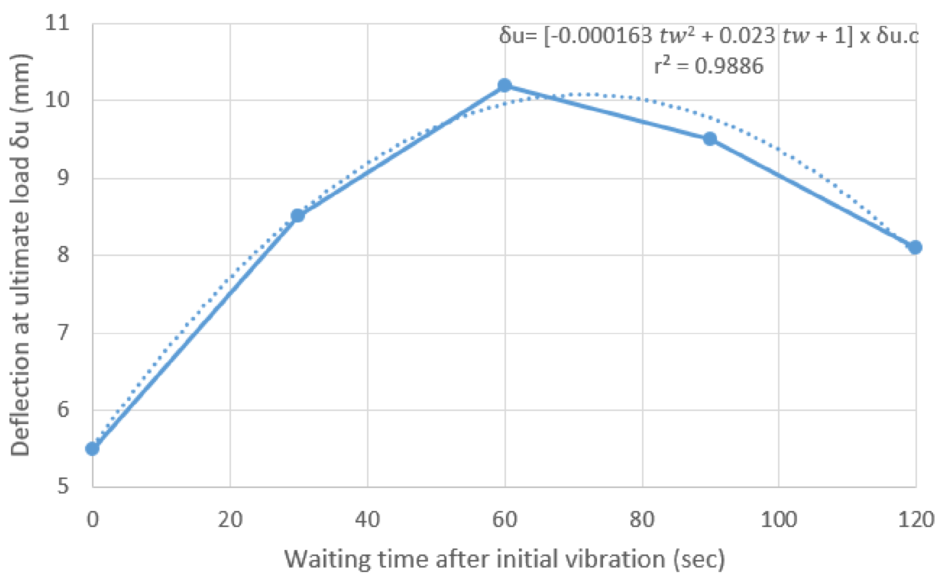

Fig. 4-35 Ultimate load deflection results of the beam with waiting time for re-vibration time length (30 sec.)

Table 4-19 and figure 4-36 compares the results of specimens (A1 to E3) in terms of their initial vibration time, waiting time after the initial vibration, re-vibration time, and deflection at the ultimate load.

Specimen A1 serves as the reference point, with an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and a deflection of 5.5 mm at the ultimate load. As the waiting time after the initial vibration increases, specimens B3, C3, D3, and E3 demonstrate varying deflection values at the ultimate load. Specimen B3, with a waiting time of 30 minutes and a re-vibration time of 45 seconds, exhibits a deflection of 10.2 mm. Specimen C3, with a waiting time of 60 minutes and the same re-vibration time, shows a significant increase in deflection to 13.35 mm. Specimen D3, with a waiting time of 90 minutes, has a deflection of 10.2 mm, the same as Specimen B3. Lastly, Specimen E3, with a waiting time of 120 minutes, displays a deflection of 8.5 mm.

In summary, the results suggest that increasing the waiting time after the initial vibration can influence the deflection at the ultimate load. Specimen C3, with the longest waiting time, shows the highest deflection, indicating increased structural flexibility.

Table 4-19 Strain and Deflection results of the beam for different waiting time

No	Specimen	Initial Vibration time (sec)	w_t after initial vibration (min)	t_{rv} (sec)	Strain	δu (mm)	P_{cr} (kN)	δ_{cr}	Stiffness $k = \frac{P_{cr}}{\delta_{cr}}$
1	A1	15	0	0	0.001726	5.5	42	1.2	35
2	B3	15	30	45	0.002085	10.2	49	1.5	32.66
3	C3	15	60	45	0.002987	13.35	55	2.05	26.82
4	D3	15	90	45	0.001704	10.2	50	2.3	21.73
5	E3	15	120	45	0.002073	8.5	49	2.3	21.3

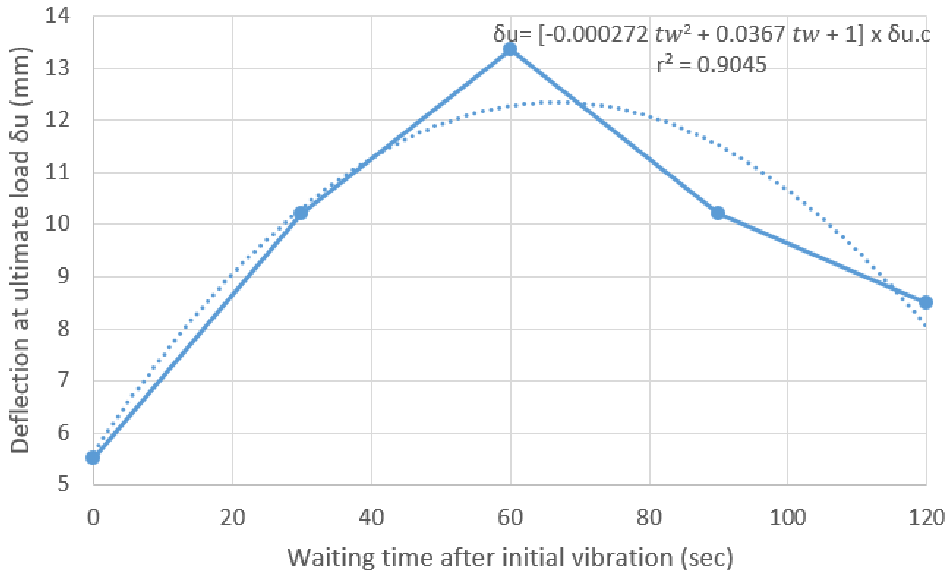

Fig. 4-36 Ultimate load deflection results of the beam with waiting time for re-vibration time length (45 sec.)

Table 4-20 and figure 4-37 compares the results of specimens (A1 to E4) in terms of their initial vibration time, waiting time after the initial vibration, re-vibration time, and deflection at the ultimate load. Specimen A1 serves as the reference point, with an initial vibration time of 15 seconds, no waiting time after the initial vibration, no re-vibration time, and a deflection of 5.5 mm at the ultimate load.

As the waiting time after the initial vibration increases, specimens B4, C4, D4, and E4 demonstrate varying deflection values at the ultimate load. Specimen B4, with a waiting time of 30 minutes and a re-vibration time of 60 seconds, exhibits a deflection of 7.78 mm. Specimen C4, with a waiting time of 60 minutes and the same re-vibration time, shows a slightly higher deflection of 8.5 mm. Specimen D4, with a waiting time of 90 minutes, has a deflection of 7.8 mm, similar to Specimen B4. Lastly, Specimen E4, with a waiting time of 120 minutes, displays the lowest deflection value of 6.9 mm.

In summary, the results suggest that increasing the waiting time after the initial vibration can influence the deflection at the ultimate load. Specimen C4, with the longest waiting time, shows the highest deflection, indicating increased structural flexibility.

Table 4-20 Strain and Deflection results of the beam for different waiting time

No	Specimen	Initial Vibra-tion time (sec)	w_t after initial vibration (min)	t_{rv} (sec)	Strain	δu (mm)	P_{cr} (KN)	δ_{cr}	Stiff-ness $k = \frac{P_{cr}}{\delta_{cr}}$
1	A1	15	0	0	0.001726	5.5	42	1.2	35
2	B4	15	30	60	0.001415	7.78	46	2.3	20
3	C4	15	60	60	0.001752	8.5	53	2.75	19.27
4	D4	15	90	60	0	7.8	47	2	23.5
5	E4	15	120	60	0	6.9	43	1.8	23.88

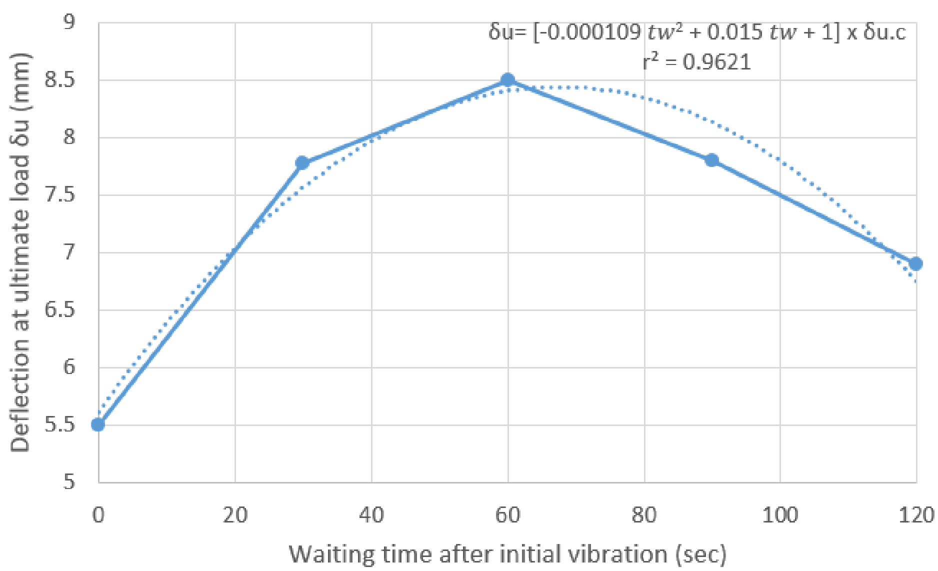

Fig. 4-37 Ultimate load deflection results of the beam with waiting time for re-vibration time length (60 sec.)

4.6 Impact of Re-vibration

This experiment was performed to evaluate the main hypotheses of the role of vibration and re-vibration with different time intervals and waiting time on the reinforced and non-reinforced concrete beams.

The results have shown that the beams with the same compressive strength and the maximum size of aggregate (12.5) the mechanical properties of concrete (compressive strength, tensile strength, flexural strength, and modulus of elasticity), strain, and deflection of concrete with various time duration and re-vibration techniques was increased for the 1st one hour and re-vibrated for 45 seconds time duration. After that for waiting time 1.5 and 2 hours before re-vibration and re-vibrated for 60 seconds was decreased.

Table 4-21 shows the comparing these specimens, it is evident that the combination of reinforcement type and vibration parameters significantly influences the first crack load P_{cr} and ultimate load Pu values. Specimens with longitudinal reinforcement along with stirrups tend to have higher first crack load P_{cr} and ultimate load P_u values compared to specimens with only stirrups or no reinforcement. Additionally, increasing the re-vibration time generally leads to higher load-bearing capacities. The absence of reinforcement results in significantly lower load-bearing capacities.

Overall, these findings highlight the importance of appropriate reinforcement and vibration parameters in enhancing the structural performance of specimens.

Table 4-21 Effect of reinforcement on first crack and ultimate loads of the beam for different re-vibration time

No	Specimen	Initial Vibration time (sec)	w_t after initial vibration (min)	t_{rv} (sec)	Reinforcement	P_{cr} (kN)	P_u (kN)
1	A1	15	0	0	longitudinal + stirrups	42	102
2	C1	15	60	15	longitudinal + stirrups	49	107
3	F1	15	60	15	stirrups	41	73.8
4	GI	15	60	15	without reinforcement		11.9
5	A2	30	0	0	longitudinal + stirrups	44	104
6	C2	15	60	30	longitudinal + stirrups	51	114
7	F2	15	60	30	stirrups	43	78
8	G2	15	60	30	without reinforcement		14.5
9	A3	45	0	0	longitudinal + stirrups	45	109
10	C3	15	60	45	longitudinal + stirrups	55	119
11	F3	15	60	45	stirrups	47	80
12	G3	15	60	45	without reinforcement		18.7
13	A4	60	0	0	longitudinal + stirrups	47	100
14	C4	15	60	60	longitudinal + stirrups	53	111
15	F4	15	60	60	stirrups	42	70
16	G4	15	60	60	without reinforcement	9.5	9.5

CHAPTER FIVE

THEORETICAL CALCULATION

5.1 Shear strength calculation

The shear strength was calculated using theoretical and empirical methods found in codes and literature. The shear strength calculated for the re-vibrated beams with different waiting times, the methods used to calculate the shear strength of concrete beams are described below:

5.1.1 ACI Code 318-19

$$V_c = \frac{\lambda}{6}\sqrt{f_c'}\,b_w d = \frac{P_{cr}}{2} \qquad \text{..........5-1}$$

$P_{cr} = 2\ V_c$

$$V_c = \frac{2}{3}\lambda\rho^{\frac{1}{3}}\sqrt{f_c'}\,b_w d \qquad \text{.........5-2}$$

$V_s = \frac{A_v\, fy\, d}{S}$ where $A_v = \frac{\pi d^2}{4}$

$V_u = V_c + V_s = \frac{P_u}{2}$

$P_u = 2\ V_u$

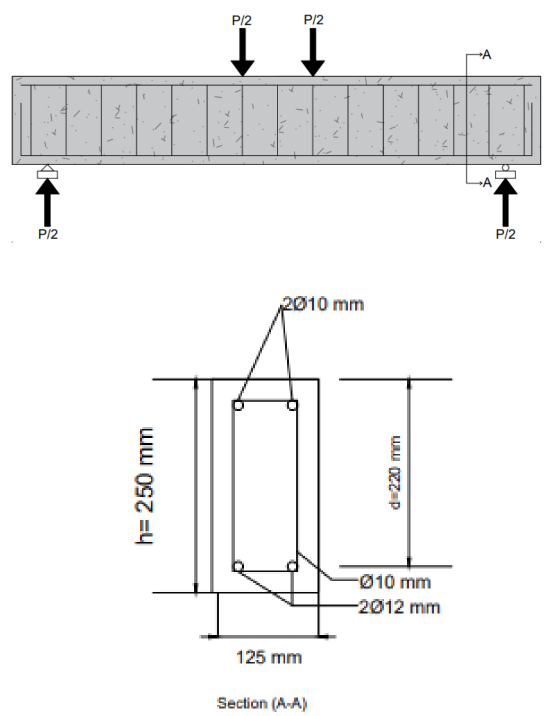

Fig. 5-1 Beam section (A-A)

Where:

V_c = Shear strength of concrete (N)

f_c' = Cylinder compressive strength of the beam (Mpa)

b_w = Width of the beam (mm)

d = effective depth of the beam (mm)

P_{cr} = First crack load (N)

A_v = Cross section area of stirrup legs ($2A_b$) (mm^2)

V_s = Shear strength of stirrups (N)

f_y= Yield strength of stirrups (Mpa)

S = Spacing between stirrups

V_u = Ultimate shear strength of the beam

P_u = Ultimate load which the beam can supported

λ = light weight concrete modification factor

λ = 1   for the concrete that has normal weight

Table 5-1 The experimental shear force and that calculated from the available code

No.	Specimen	Re-vibration time in (sec)	P_{cr} (kN)	$V_{c\ exp.}$ (kN)	$V_{c1.ACI}$ (kN) Eq.(5-1)	$R=\frac{V_{c\ exp.}}{V_{c1.ACI}}$	$V_{c2.ACI}$ (kN) Eq.(5-2)	$R=\frac{V_{c\ exp.}}{V_{c2.ACI}}$
1	A1	0	42	21	26.28	0.8	21.21	0.99
2	A2	0	44	22	26.8	0.82	21.64	1.02
3	A3	0	45	22.5	27.43	0.82	22.15	1.02
4	A4	0	47	23.5	25.49	0.92	20.59	1.14
5	B1	15	45	22.5	27.32	0.82	22.06	1.02
6	B2	30	47	23.5	27.95	0.84	22.57	1.04
7	B3	45	49	24.5	28.36	0.86	22.91	1.07

8	B4	60	46	23	26.62	0.86	21.49	1.07
9	C1	15	49	24.5	28.61	0.86	23.11	1.06
10	C2	30	51	25.5	28.9	0.88	23.34	1.09
11	C3	45	55	27.5	30.14	0.91	24.34	1.13
12	C4	60	53	26.5	28.32	0.94	22.87	1.16
13	D1	15	46	23	27.51	0.84	22.22	1.04
14	D2	30	48	24	28.25	0.85	22.82	1.05
15	D3	45	50	25	28.58	0.87	23.08	1.08
16	D4	60	47	23.5	28.69	0.82	21.56	1.09
17	E1	15	46	23	25.86	0.89	20.88	1.10
18	E2	30	48	24	26.6	0.9	21.48	1.12
19	E3	45	49	24.5	25.51	0.96	20.6	1.19
20	E4	60	43	21.5	25.43	0.85	20.54	1.05
21	F1	15	41	20.5	28.61	0.72	23.11	0.89
22	F2	30	43	21.5	28.9	0.74	23.33	0.92
23	F3	45	47	23.5	30.14	0.78	24.34	0.97
24	F4	60	42	21	28.32	0.74	22.87	0.92
R_{avg}						0.845		1.05
R_{Max}						0.96		1.19
R_{Min}						0.72		0.89
σ						0.0612		0.08
Var						0.0037		0.0058
Corr.(r)						0.354		0.352
r^2						0.125		0.124

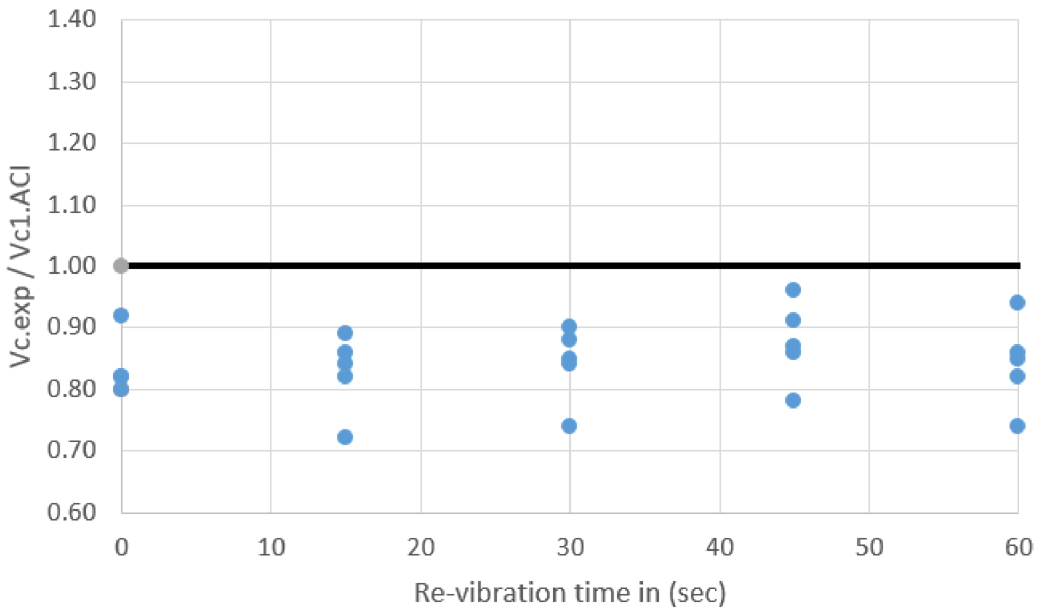

Fig. 5-2 Experimental/calculated shear force (ACI Code-19) eq. [5-1]

The table 5-1 and figure 5-2 presents data on shear strength properties of different specimens (A1 to F4) subjected to re-vibration for different time intervals (15, 30, 45, and 60 seconds). The specimens were tested for cracking load, shear strength from experimental results, shear strength calculated using the ACI (318-19) eq. 5-1, and the ratio of experimental shear strength to ACI shear strength. For specimens A1 to A4 vibrated only one time, indicating that these specimens were not re-vibrated during casting. Specimens B1 to F4 were re-vibrated at 15, 30, 45, and 60 seconds, respectively, during the testing process.

The cracking load values for all specimens range from 41 to 55 kN. As the re-vibration time increases, there is a slight increase in the cracking load. The shear strength from experimental results and the shear strength calculated using the ACI code are provided for each specimen. The ratio of experimental shear strength to ACI shear strength is also calculated and presented.

Upon examination of the data, it can be observed that as the re-vibration time increases, the shear strength tends to increase slightly. The ratio of experimental shear strength to ACI shear strength varies for different specimens but generally stays close to 0.82 to 0.94.

(R_{avg}) represents the average ratio of shear experimental/shear ACI code across all specimens, which is approximately 0.845. This suggests that, on average, the experimental shear values tend to be about 85% of the ACI code predictions.

R_{max} and R_{min} represent the maximum and minimum ratios in the dataset, which are 0.96 and 0.72, respectively. This indicates that some specimens had shear values significantly above (R_{max} or below R_{min} the ACI code predictions.

σ represents the standard deviation of the ratios, which is approximately 0.0612. This value indicates the average amount of deviation from the mean ratio and gives an idea of the spread of the data and (Var) represents the variance of the ratios, which is approximately 0.0037. This value provides insight into the variability of the experimental shear ratios around the average ratio.

Corr.(r) represents the correlation coefficient (r) between the experimental and ACI code shear ratios. The value of 0.354 indicates a moderate positive correlation between the two sets of ratios and r^2 represents the coefficient of determination, which measures the proportion of variability in the experimental shear ratios that can be explained by the ACI code ratios. The value of 0.125 suggests that only about 12.5% of the variability is explained by the ACI code.

Overall, the table shows that the experimental shear values tend to be lower than the ACI code predictions on average. There is variability among individual specimens, with some having shear values significantly different from the predictions. The correlation and coefficient of determination indicate a moderate relationship between the two sets of ratios.

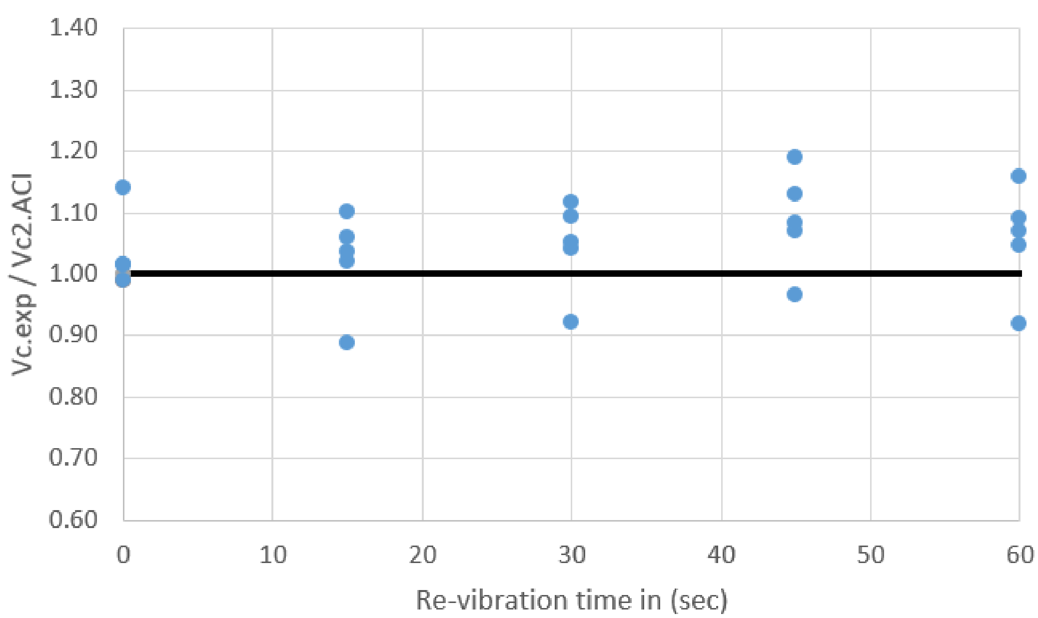

Fig. 5-3 Experimental/calculated shear force (ACI Code-19) eq. [5-2]

The figure 5-3 presents a comparison between experimental shear strengths and those predicted by the ACI code. Eq. (5-2). The statistical measures provided give insights into the average, variability, maximum, and minimum differences between the two sets of values, along with the strength and quality of the linear relationship between them. (R_{avg}) represents the average ratio of shear experimental/shear ACI code across all specimens, which is approximately 1.05. This suggests that, on average, the experimental shear values tend to be about 5% higher than the ACI code predictions.

R_{max} and R_{min} represent the maximum and minimum ratios in the dataset, which are 1.19 and 0.89, respectively. This indicates that some specimens had shear values significantly below R_{max} or above R_{min} the ACI code predictions.
The standard deviation (σ) of the ratios, which is approximately 0.08 This value indicates the average amount of deviation from the mean ratio and gives an idea of the spread of the data.

And (Var) represents the variance of the ratios, which is approximately 0.0058. Variance measures the spread of data points from the average and provides insight into the variability of the experimental shear ratios.

(r) represents the correlation coefficient between the experimental and ACI code shear ratios. The value of 0.352 indicates a moderate positive correlation between the two sets of ratios and r^2 represents the coefficient of determination, which measures the proportion of variability in the experimental shear ratios that can be explained by the ACI code ratios. The value of 0.124 suggests that only about 12.4% of the variability is explained by the ACI code.

5.2 Flexural calculations

Flexural calculations of beams involve determining the bending stresses and deflections that occur when a beam is subjected to applied loads. The main goal of these calculations is to ensure that the beam can safely support the applied loads without failure or excessive deflection. This process is essential in structural engineering and design to ensure the structural integrity and safety of buildings, bridges, and other structures. Nominal bending moment of rectangular section shown in figure (5-4), can be calculated from the following equation for under reinforcement concrete beam

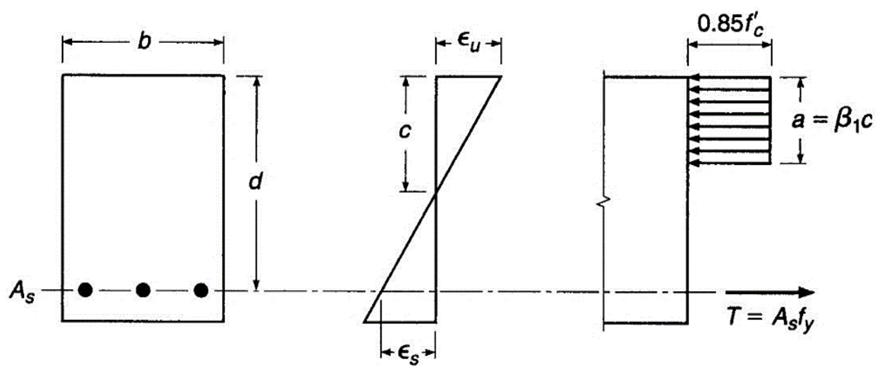

Fig. 5- 4 Singly reinforced concrete beam

5.2.1 Ultimate moment capacity:

$$\mathrm{M_n} = \mathrm{A_s}\ \mathrm{f_y}(d - \frac{a}{2}) \quad (5\text{-}3)$$

$$a = \frac{As\ fy}{0.85\ fc\prime b_w} \quad (5\text{-}4)$$

$$\mathrm{M_n} = \rho b d^2 f y\left(1 - 0.59\frac{\rho f y}{f_c'}\right) \quad (5\text{-}5)$$

$$\rho = \frac{\mathrm{A_s}}{bd} \quad (5\text{-}6)$$

$$\mathrm{M_{n\ cal.}} = \frac{P_{\mathrm{n\ cal.}}}{2} \times a \quad \text{or} \quad P_{\mathrm{n\ cal.}} = \frac{2\ \mathrm{M_{n\ cal.}}}{a} \quad (5\text{-}7)$$

Where:

P_n = nominal failure load (N)

a = Shear span (mm)

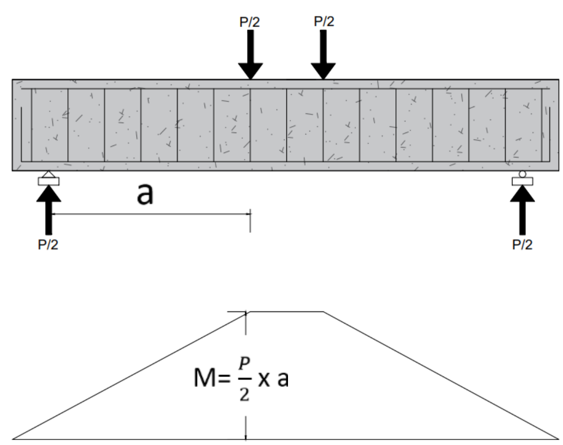

Fig. 5-5 Beam layout and moment diagram

5.2.2 Cracking bending moment:

$$M_{cr} = \frac{P_{cr}}{2} \times a \tag{5-8}$$

Where:

$P_{cr} =$ Cracking load (N)

$$y' = \frac{bh(\frac{h}{2})+(n-1)As\ d+(n-1)As'd'}{bh+(n-1)As+(n-1)As'} \tag{5-9}$$

A_s = area of reinforcement in tension zone (mm^2)

A_s'= area of reinforcement in compression zone (mm^2)

Ig = Moment of inertia of the transformed section

$$I_g = \frac{b_w h^3}{12} + b_w h\,(y' - \frac{h}{2})^2 + (n-1)A_s\,(d-y')^2 + \left(n'-1\right)A_s'(y'-d')^2 \tag{5-10}$$

$$M_{cr} = \frac{f_r\, I_g}{y_t} \tag{5-11}$$

Where: $f_r =$ modulus of rupture of the concrete $= 0.625\sqrt{f_c'}$ (MPa)

$y_t = h - y'$ distance from the neutral axes to the tension face (mm)

$$P_{cr} = \frac{2M_{cr}}{a} \tag{5-12}$$

The theoretical and experimental cracking and ultimate bending moments of all beams are shown in table 5-2, generally the experimental bending moment is greater than theoretical values, and the ratio of ($M_{n\ exp.}/M_{n\ cal.}$) of all beams are more than one, the average ratio (1.082).

This table presents data related to the bending moments of different specimens subjected to re-vibration and varying waiting times after the initial vibration. The goal is to compare the bending moments obtained from the experimental tests with those predicted by the ACI eq. (5-5)

Each row represents a specific test specimen identified by a label (A1 to E4), the waiting time after initial vibration indicates the time interval after the initial vibration when the specimen was tested. The specimens were tested at different waiting times, ranging from 30 to 120 minutes.

The re-vibration time is the duration of additional vibration applied to the specimen in seconds before testing. The specimens were re-vibrated for different durations.

The bending moment values calculated using the ACI equation, which is a standard formula for computing bending moments in reinforced concrete elements and bending moment values obtained through experimental testing of each specimen. the ratio of the bending moment obtained from experimental testing to the moment calculated using the ACI equation. This ratio allows for a direct comparison between the experimental and predicted values, showing how closely the experimental results align with the ACI predictions.

In summary, this table offers a comprehensive comparison between the bending moments obtained from experimental tests and the moments calculated using the ACI equation for different specimens subjected to various waiting times and re-vibration durations. The ratio of experimental to ACI moments gives valuable insights into the accuracy of the ACI equation in predicting the bending behavior of the tested specimens under the given conditions.

Table 5-2 Experimental and calculated moment ratio with cracking moment

Specimen	Waiting time after initial vibration t_w (min)	Re-vibration Time t_{rv} (Sec)	Mn ACI eq. (5-5) kN.m	$M_{\mathrm{n\ exp.}}$ kN.m	$M_{\mathrm{n\ exp.}}/M_{\mathrm{n\ cal.}}$
A1	0	0	28.353	29.07	1.025
B1	30	15	28.571	29.64	1.037
B2	30	30	28.69	32.205	1.123
B3	30	45	28.764	33	1.147
B4	30	60	28.428	30.5	1.073
C1	60	15	28.806	30.5	1.059
C2	60	30	28.855	32.5	1.126
C3	60	45	29.048	33.9	1.167
C4	60	60	28.756	31.5	1.095
D1	90	15	28.607	29	1.014
D2	90	30	28.744	31.2	1.085
D3	90	45	28.8	32	1.111
D4	90	60	28.443	30.5	1.072
E1	120	15	28.258	29	1.026
E2	120	30	28.424	30.5	1.073
E3	120	45	28.176	31	1.100
E4	120	60	28.157	30	1.065
R_{avg}					1.082

R_{Max}		1.167
R_{Min}		1.014
Σ		0.044
Var		0.002
Corr.(r)		0.708
r^2		0.501

5.3 Proposed bending moment equations

The ACI moment equation for beams is modified for the re-vibrated beams as a function of the variables taken into consideration in this study: Initial vibration, re-vibration time and waiting time after vibration. The best fit curve is determined using linear regression analysis and applied on the experimental data, the following equation is proposed to predict the moment equation of the re-vibrated beams in term of different variables:

$$M_o = 0.89\ M_n(1 - 0.0025 * t_{rv})\left(\frac{{t_{rv}}^{0.1}}{{t_w}^{0.01}}\right) \quad (5\text{-}13)$$

Where: t_{rv} = Re-vibration time (sec)

t_w = Waiting time after initial vibration (min)

M_n = Theoretical bending moment eq. (5-5) kN.m

Table 5-3 and figure 5-6 presents a comparison of bending moments obtained from experimental testing ($M_{\text{n.exp.}}$) with those predicted using the ACI (American Concrete Institute) equation (Mn ACI) for various test specimens. The table also includes other relevant parameters like waiting time (t_w) after initial vibration, re-vibration time (t_{rv}) in seconds, applied load, equation used for prediction, difference between the predicted and practical moments, and accuracy.

Table 5-3 Summary of statistical data and proposed equation

No	Specimen	t_w (min)	Re-vibration Time t_{rv} (Sec)	$M_{n.c}$ (kN.m)	$M_{n.exp}$ (kN.m)	Load (kN)	$M_{n.ACI}$ eq. (5-5) (kN.m)	R= $\frac{M_{n.exp}}{M_{n.ACI}}$	$R_{max.}$	$R_{min.}$	$R_{avg.}$	r	r^2
1	A1	0	0	28.35	29.07	102							
2	B1	30	15	28.35	29.64	104	30.78	0.963	1.042	0.963	0.999	0.954	0.910
3	B2	30	30	28.35	32.205	113	31.70	1.015					
4	B3	30	45	28.35	33	116	31.67	1.041					
5	B4	30	60	28.35	30.5	107	31.22	0.976					
6	C1	60	15	28.35	30.5	107	30.56	0.997	1.078	0.998	1.031	0.903	0.815
7	C2	60	30	28.35	32.5	114	31.48	1.032					
8	C3	60	45	28.35	33.9	119	31.46	1.077					
9	C4	60	60	28.35	31.5	111	31.01	1.015					
10	D1	90	15	28.35	29	102	30.44	0.952	1.021	0.953	0.989	0.956	0.914
11	D2	90	30	28.35	31.2	109	31.35	0.995					
12	D3	90	45	28.35	32	112	31.33	1.021					

13	D4	90	60	28.35	30.5	107	30.88	0.987					
14	E1	120	15	28.35	29	102	30.35	0.955	0.992	0.955	0.974	0.961	0.924
15	E2	120	30	28.35	30.5	107	31.26	0.975					
16	E3	120	45	28.35	31	109	31.24	0.992					
17	E4	120	60	28.35	30	105	30.79	0.974					

Where: $M_{n.c}$ = Bending moment of control beam

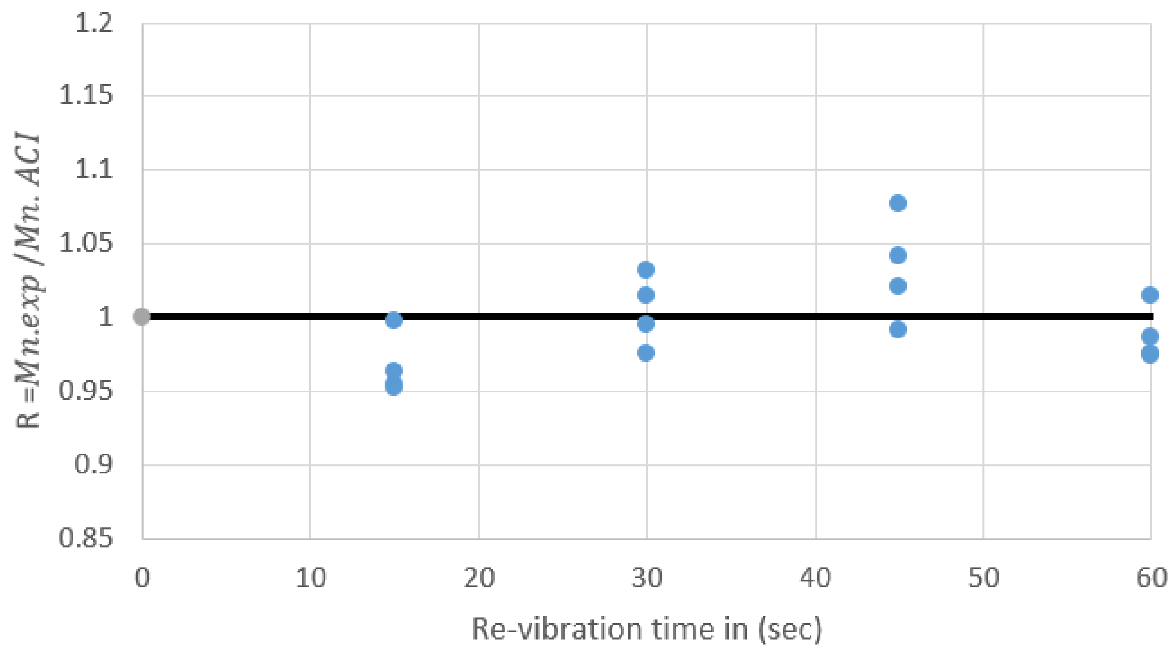

Fig. 5-6 Relation of re-vibration time and R ($\frac{M_{n.exp}}{M_{n.ACI}}$)

For most specimens, the practical moment ($M_{n.exp}$) is higher than the ACI-predicted moment ($M_{n.ACI}$), as indicated by the average ratios being greater than 1, and the accuracy of the predictions is generally high, with values close to 1, indicating a reasonably good fit between the practical and ACI-predicted moments. The maximum and minimum ratios (R_{max} and R_{min}) give an idea of the range of accuracy in load predictions across different specimens.

The coefficient of correlation (r) is close to 1, indicating a strong positive correlation between the experimental and ACI-predicted bending moments.

Overall, the table suggests that the ACI equation provides reasonably accurate predictions for the bending moments of the tested specimens, and there is a consistent correlation between the experimental and predicted moments. However, further investigation and verification of the missing data or calculation errors are necessary for a complete and reliable analysis.

5.4 Proposed concrete compressive strength equation ($f_c^{'}$)

A second degree polynomial equation is proposed to show the effect of waiting time (t_w) after the re-vibration of concrete. Initial vibration is (15 sec) and the re-vibration times (t_{rv}) are equal to (15, 30, 45 and 60 sec) for different waiting time (30, 60, 90 and 120 minute), as shown in table 5-4 and figure 5-7 till 5-15.

Table 5-4 Proposed equations for compressive strength (f_c')

No.	Re-vibration time t_{rv} (sec)	Proposed equation	f_c' practical	f_c' equation	Ratio	R_{Max}	R_{Min}	R_{avg}	σ	Var	r	r^2
1	15	f_c' = [-0.0000486 t_w^2 + 0.00567 t_w + 1] x $f'_{c.c}$ Eq. (5-14)	32.873	32.873	1	1.018	0.960	0.990	0.0213	0.0005	0.959	0.920
			35.556	37.025	0.960							
			38.98	38.297	1.018							
			36.036	36.689	0.982							
			31.836	32.201	0.989							
2	30	f_c' =[-0.0000517 t_w^2 + 0.0065 t_w + 1] x $f'_{c.c}$ Eq. (5-15)	32.873	32.873	1	1.004	0.985	0.994	0.0079	0.0001	0.995	0.990
			37.213	37.763	0.985							
			39.766	39.593	1.004							
			38.013	38.363	0.991							
			33.703	34.073	0.989							
3	45	f_c' = [-0.0000669 t_w^2 + 0.0085 t_w + 1] x $f'_{c.c}$ Eq. (5-16)	32.873	32.873	1	1.038	0.969	1.000	0.0289	0.0008	0.955	0.913
			38.313	39.254	0.976							
			43.256	41.675	1.038							
			38.886	40.136	0.969							
			35.3	34.637	1.019							
4	60	f_c' = [-0.0000395 t_w^2 + 0.0044 t_w + 1] x $f'_{c.c}$ Eq. (5-17)	32.873	32.873	1	1.033	0.935	0.979	0.0385	0.0015	0.858	0.737
			33.75	36.089	0.935							
			38.2	36.965	1.033							
			33.93	35.501	0.956							
			30.793	31.697	0.971							

5	General Equation	$f_c' = \{0.03[\left(\frac{t_{rv}^{0.66}}{t_w^{0.31}}\right) \text{x}(1.9 - 0.025 * t_{rv}) + 1\}f'_{c.c}$ Eq. (5-18)	32.873	32.873	1	1.216	0.900	1.030	0.081	0.0066	0.544	0.296
			35.556	36.046	0.986							
			37.213	36.654	1.015							
			38.313	36.203	1.058							
			33.75	34.951	0.966							
			38.98	35.433	1.100							
			39.766	35.923	1.107							
			43.256	35.559	1.216							
			38.2	34.549	1.106							
			36.036	35.13	1.026							
			38.013	35.563	1.069							
			38.886	35.242	1.103							
			33.93	34.351	0.988							
			31.836	34.938	0.911							
			33.703	35.333	0.954							
			35.3	35.04	1.007							
			30.793	34.225	0.900							

In this table above the general case is a general equation to predict the concrete compression strength ($f_c^{'}$) in the term of both re-vibration times and waiting time using multilinear regression analysis.

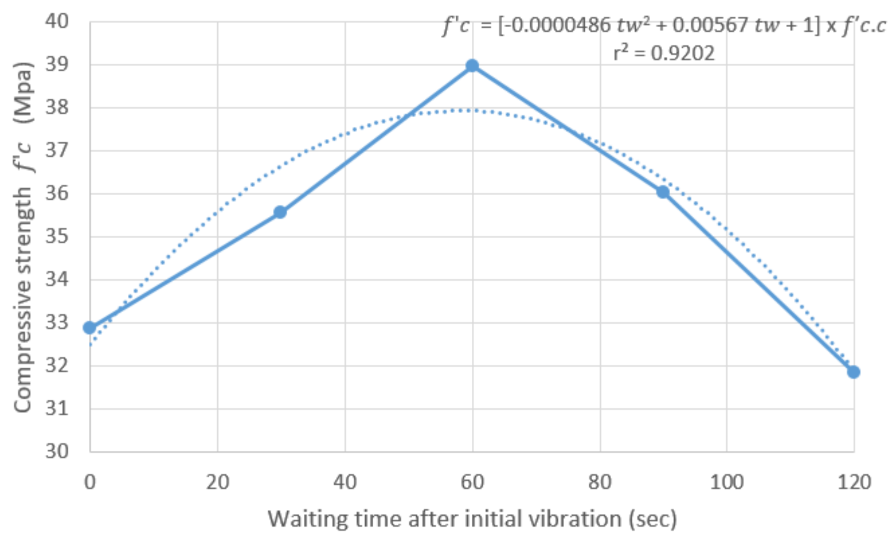

Fig. 5-7 Compressive strengths of concrete with re-vibration time (15 sec) with different waiting time eq. (5-14)

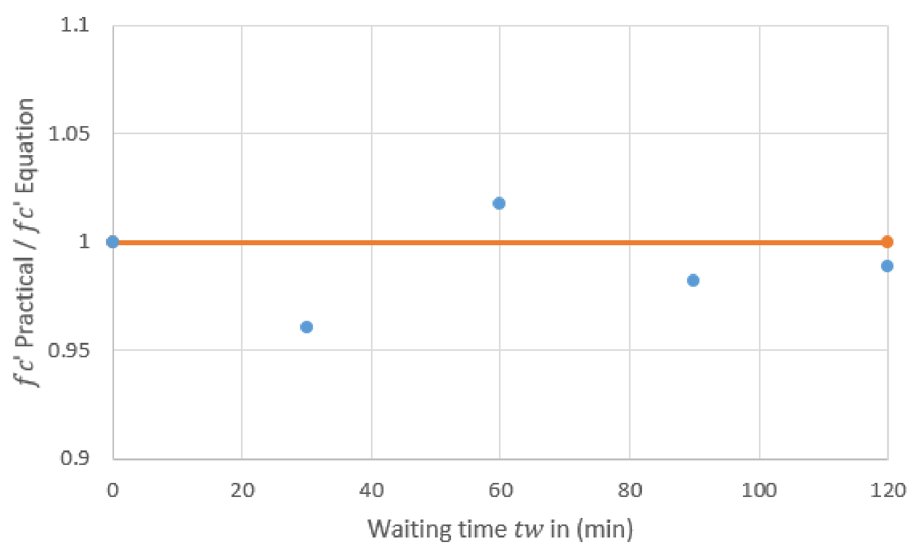

Fig. 5-8 Relative compressive strength of concrete with re-vibration time (15 sec)

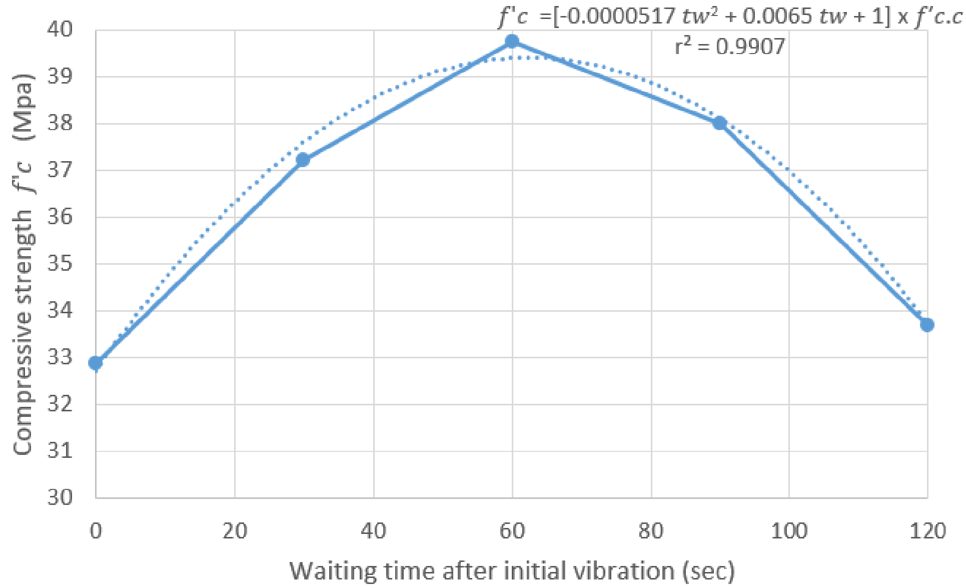

Fig. 5-9 Compressive strengths of concrete with re-vibration time (30 sec) with different waiting time eq. (5-15)

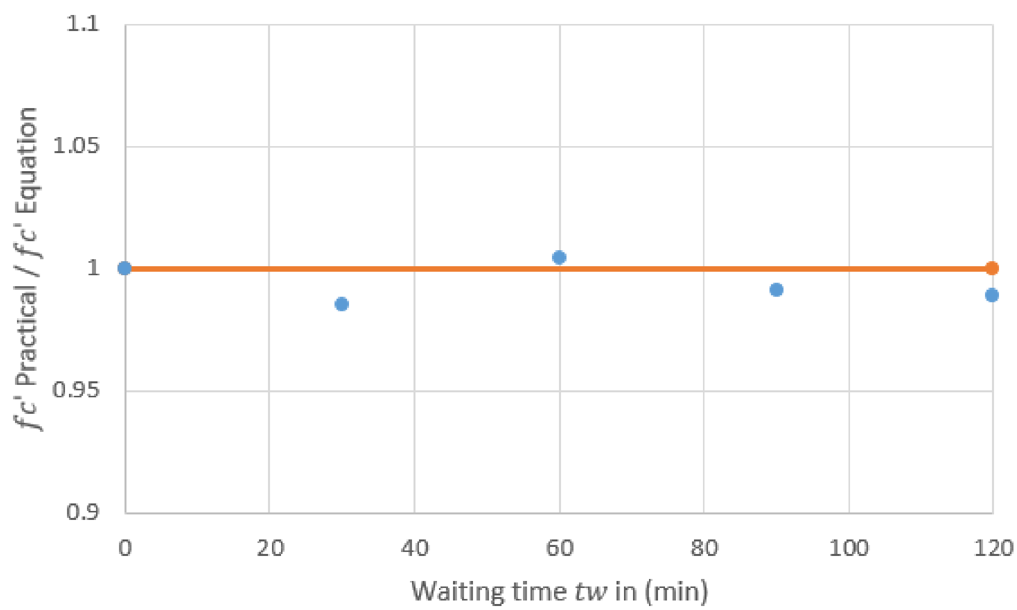

Fig. 5-10 Relative compressive strengths of concrete with re-vibration time (30 sec)

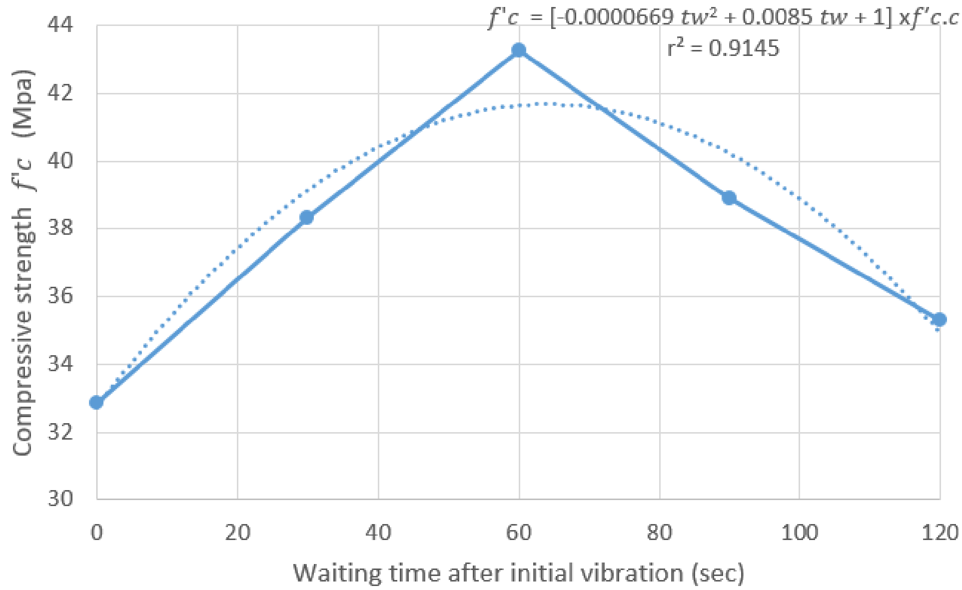

Fig. 5-11 Compressive strengths of concrete with re-vibration time (45 sec) with different waiting time eq. (5-16)

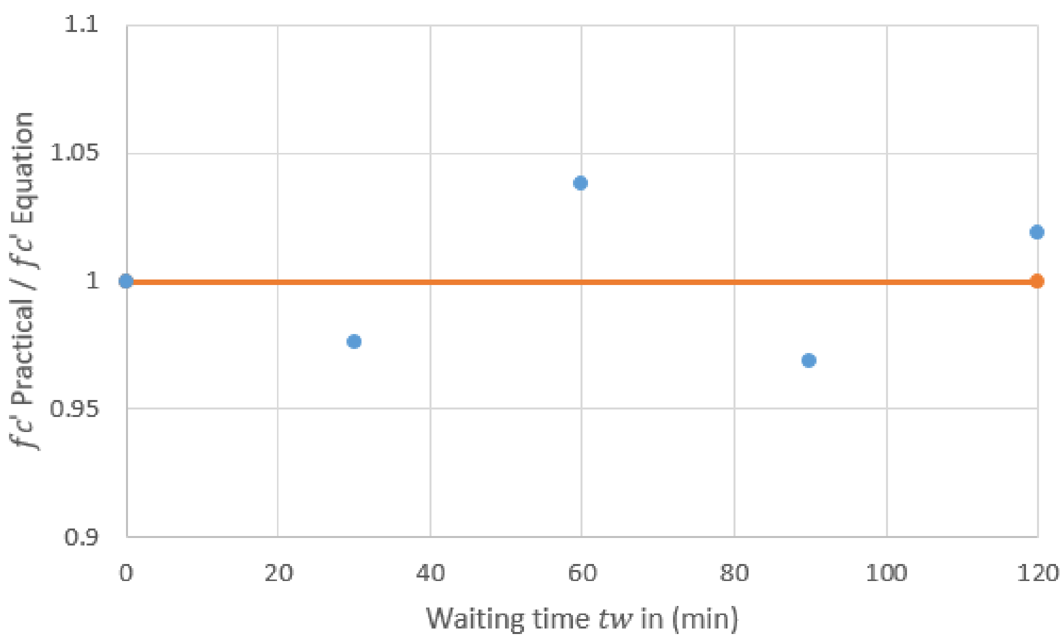

Fig. 5-12 Relative compressive strengths of concrete with re-vibration time (45 sec)

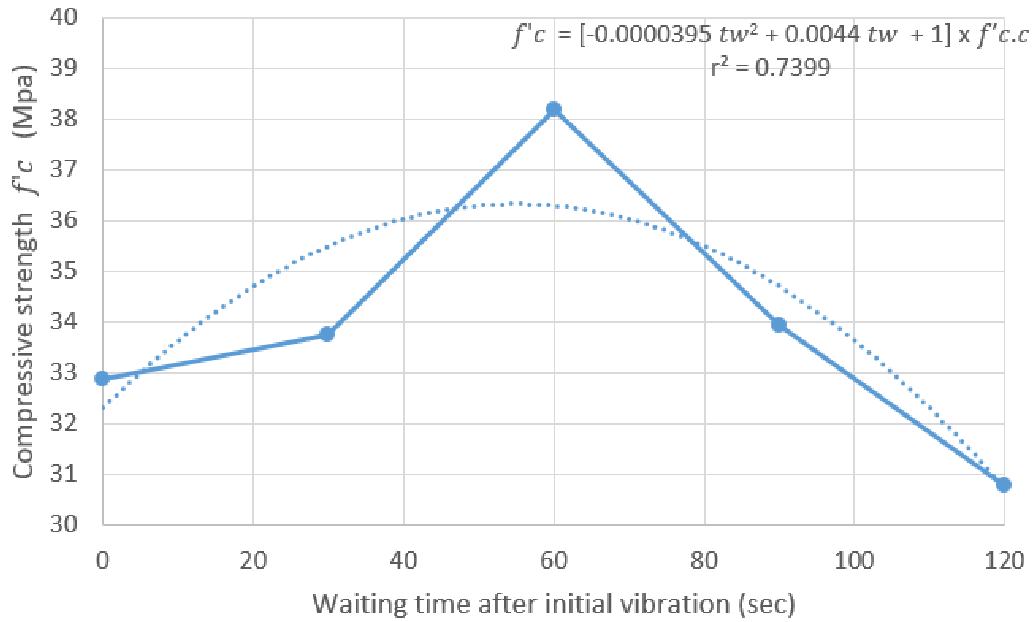

Fig. 5-13 Compressive strengths of concrete with re-vibration time (60 sec) with different waiting time eq. (5-17)

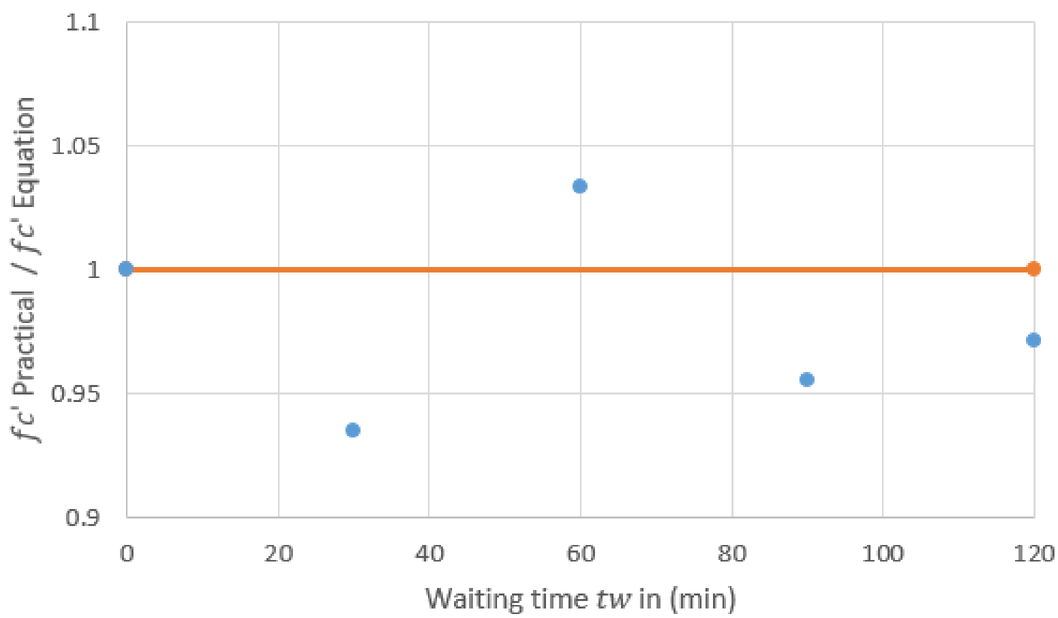

Fig. 5-14 Relative compressive strengths of concrete with re-vibration time (60 sec)

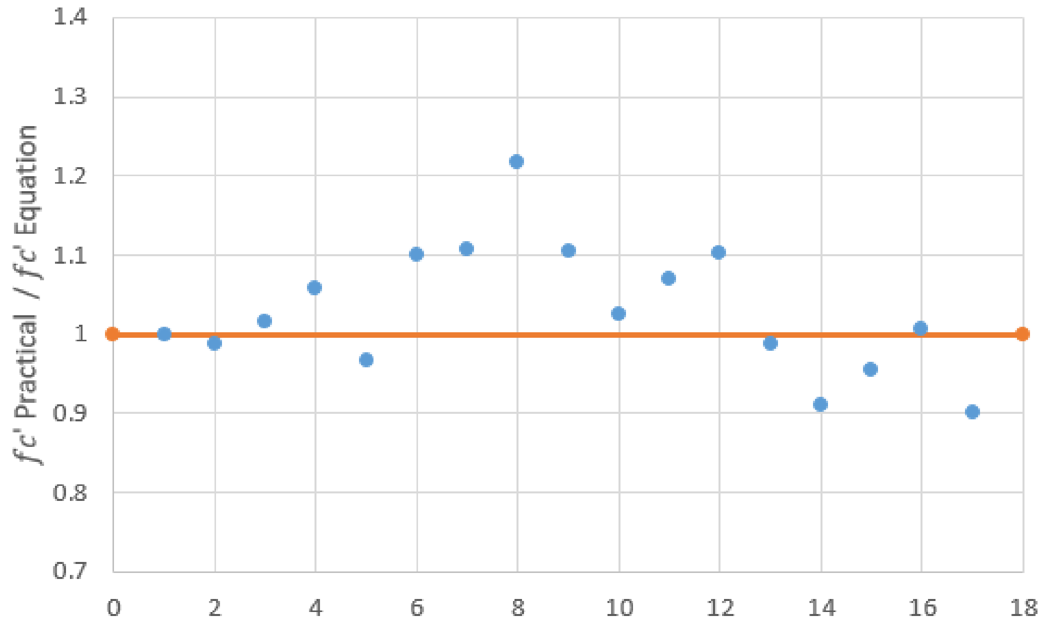

Fig. 5-15 Relative for proposed compressive strength for general equation (5-18)

5.5 Proposed concrete splitting tensile strength equation (f_{sp})

A second degree polynomial equation is proposed to show the effect of waiting time (t_w) after the re-vibration of concrete. Initial vibration is (15 sec) and the re-vibration times (t_{rv}) are equal to (15, 30, 45 and 60 sec) for different waiting time (30, 60, 90 and 120 minute), as shown in table 5-5 and figure 5-16 till 5-24.

Table 5-5 Proposed equations for splitting tensile strength (f_{sp})

No	t_{rv} (sec)	Proposed equation	f_{sp} Experi-mental (Mpa)	f_{sp} Theo-ritical (Mpa)	Ratio	R_{Max}	R_{Min}	R_{avg}	σ	Var	r	r^2
1	15	f_{sp}= [-0.000031 $t_w{}^2$ + 0.00369 t_w + 1] x $f_{sp.c}$ Eq. (5-19)	3.22	3.22	1	1.041	0.908	0.978	0.0496	0.0025	0.755	0.570
			3.43	3.487	0.984							
			3.72	3.574	1.041							
			3.16	3.481	0.908							
			3.07	3.208	0.957							
2	30	f_{sp}= [-0.000031 $t_w{}^2$ + 0.00478 t_w + 1] x $f_{sp.c}$ Eq. (5-20)	3.22	3.22	1	1.011	0.906	0.964	0.0499	0.0025	0.757	0.573
			3.58	3.6177	0.990							
			3.85	3.8097	1.011							
			3.31	3.6217	0.914							
			3.22	3.5537	0.906							
3	45	f_{sp}= [-0.000062 $t_w{}^2$ + 0.008 t_w + 1] $f_{sp.c}$ Eq. (5-21)	3.22	3.22	1	1.072	0.913	0.986	0.0602	0.0036	0.826	0.682
			3.62	3.814	0.949							
			4.34	4.048	1.072							
			3.58	3.922	0.913							
			3.43	3.436	0.998							
4	60	f_{sp}= [-0.0000248 $t_w{}^2$ + 0.00186 t_w + 1] $f_{sp.c}$ Eq. (5-22)	3.22	3.22	1	1.002	0.953	0.982	0.0222	0.0005	0.956	0.914
			3.17	3.328	0.953							
			3.3	3.292	1.002							
			3.08	3.112	0.990							
			2.69	2.788	0.965							
			3.22	3.22	1	1.291	0.835	1.008	0.100	0.0101	0.434	0.188

5	General Equation	$f_{sp} = \left[0.37 * \left(\frac{t_{rv}^{0.45}}{t_w^{0.6}}\right) - 0.0062 * t_{rv} * \left(\frac{t_{rv}^{0.45}}{t_w^{0.6}}\right) + 1 \right] f_{sp.c}$ Eq. (5-23)	3.43	3.615	0.949							
			3.58	3.58	1.000							
			3.62	3.436	1.054							
			3.17	3.22	0.984							
			3.72	3.48	1.069							
			3.85	3.45	1.116							
			4.34	3.362	1.291							
			3.3	3.22	1.025							
			3.16	3.424	0.923							
			3.31	3.406	0.972							
			3.58	3.331	1.075							
			3.08	3.22	0.957							
			3.07	3.39	0.906							
			3.22	3.376	0.954							
			3.43	3.314	1.035							
			2.69	3.22	0.835							

In this table above the general case is a general equation to predict the concrete splitting tensile strength (f_{sp}) in the term of both re-vibration times and waiting time using multilinear regression analysis.

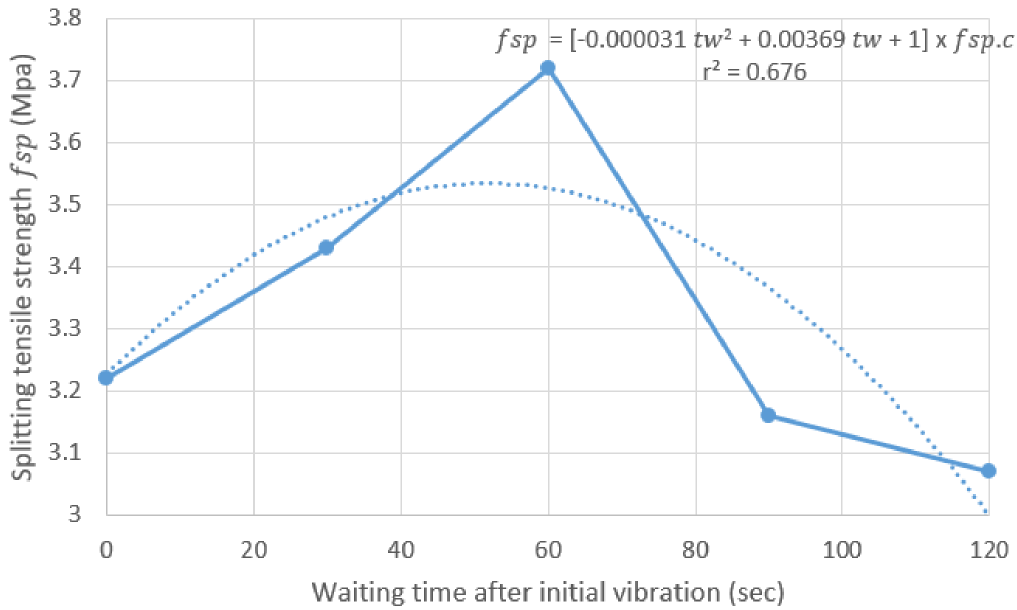

Fig. 5-16 Splitting tensile strengths of concrete with re-vibration time (15 sec) with different waiting time eq. (5-19)

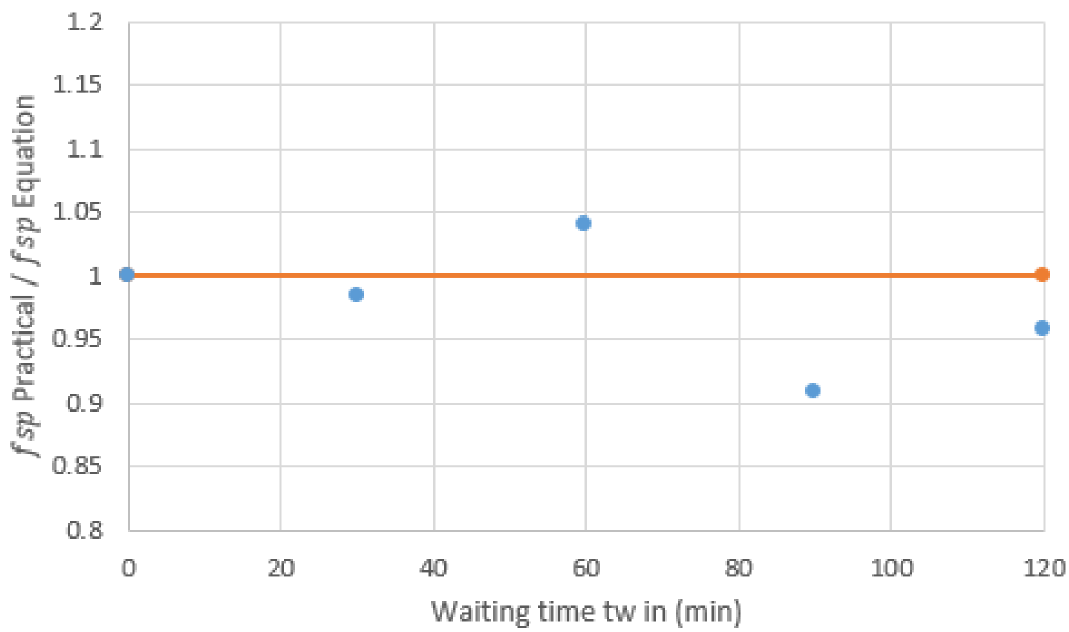

Fig. 5-17 Relative splitting tensile strengths of concrete with re-vibration time (15 sec)

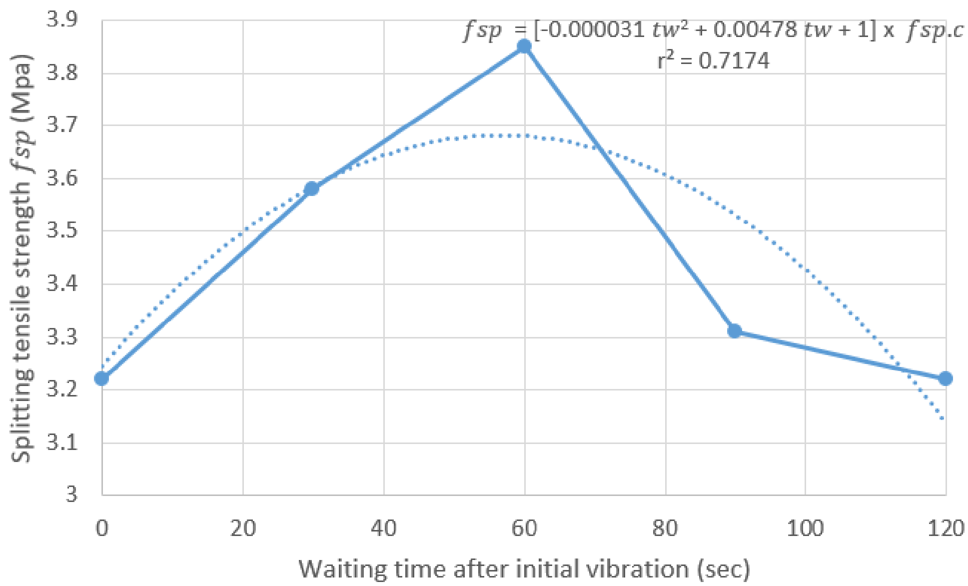

Fig. 5-18 Splitting tensile strengths of concrete with re-vibration time (30 sec) with different waiting time eq. (5-20)

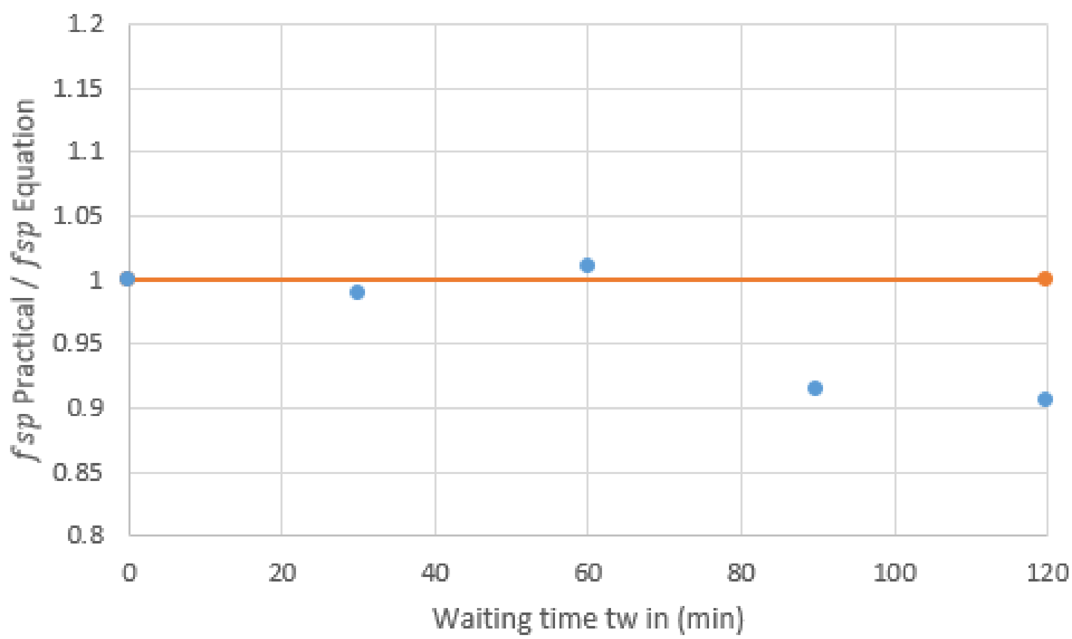

Fig. 5-19 Relative splitting tensile strengths of concrete with re-vibration time (30 sec)

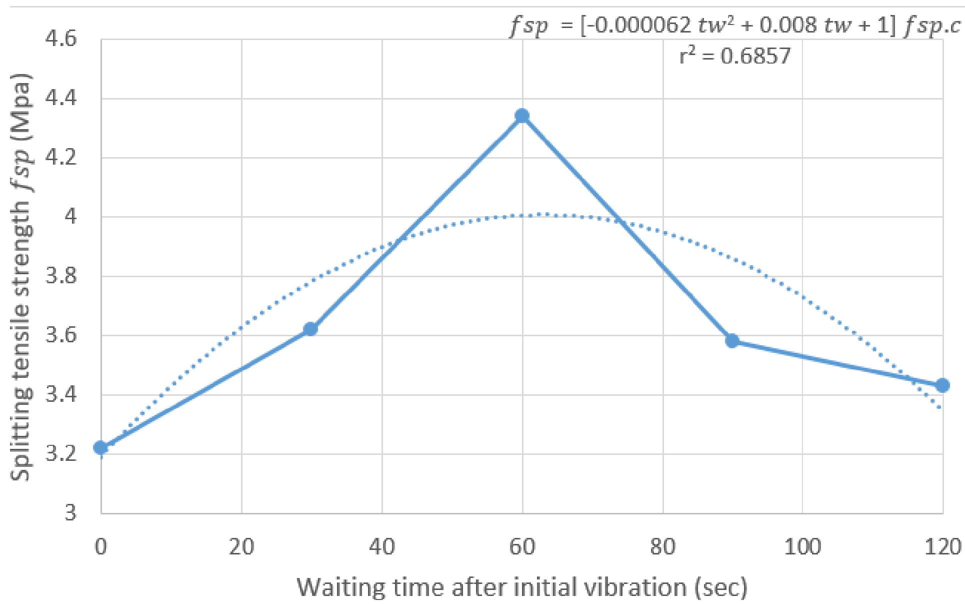

Fig. 5-20 Splitting tensile strengths of concrete with re-vibration time (45 sec) with different waiting time eq. (5-21)

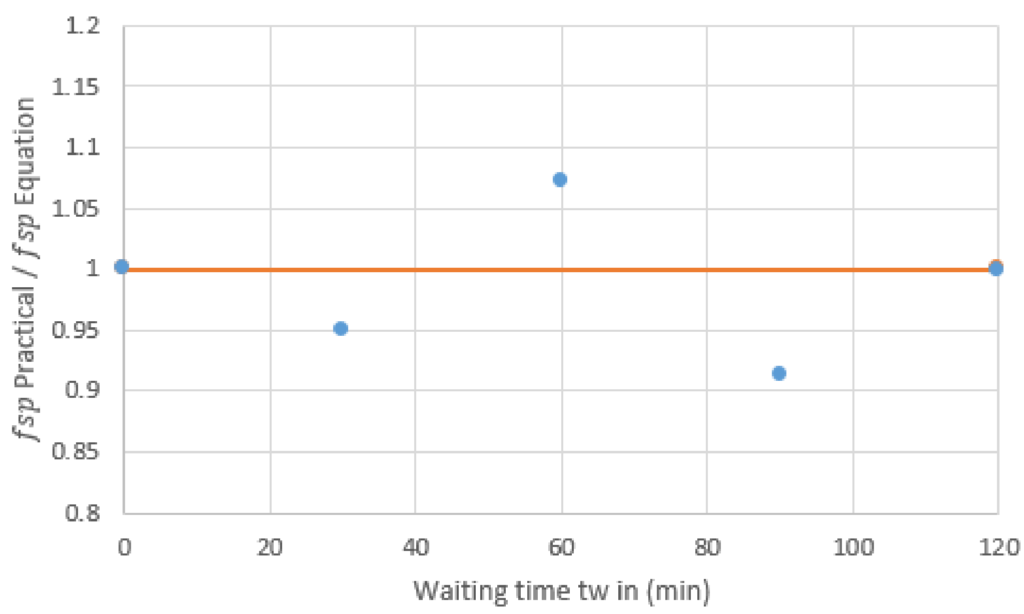

Fig. 5-21 Relative splitting tensile strengths of concrete with re-vibration time (45 sec)

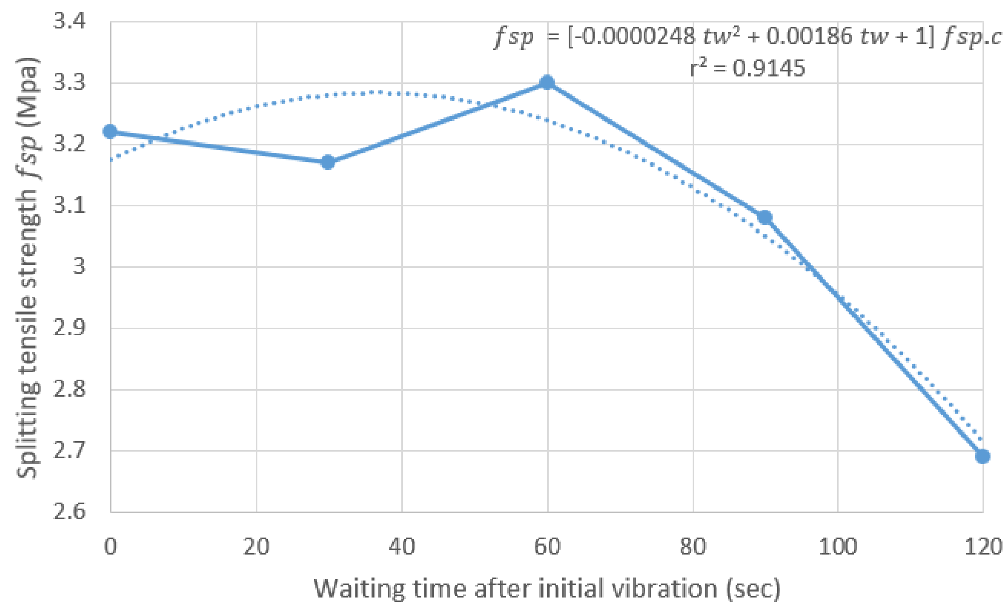

Fig. 5-22 Splitting tensile strengths of concrete with re-vibration time (60 sec) with different waiting time eq. (5-22)

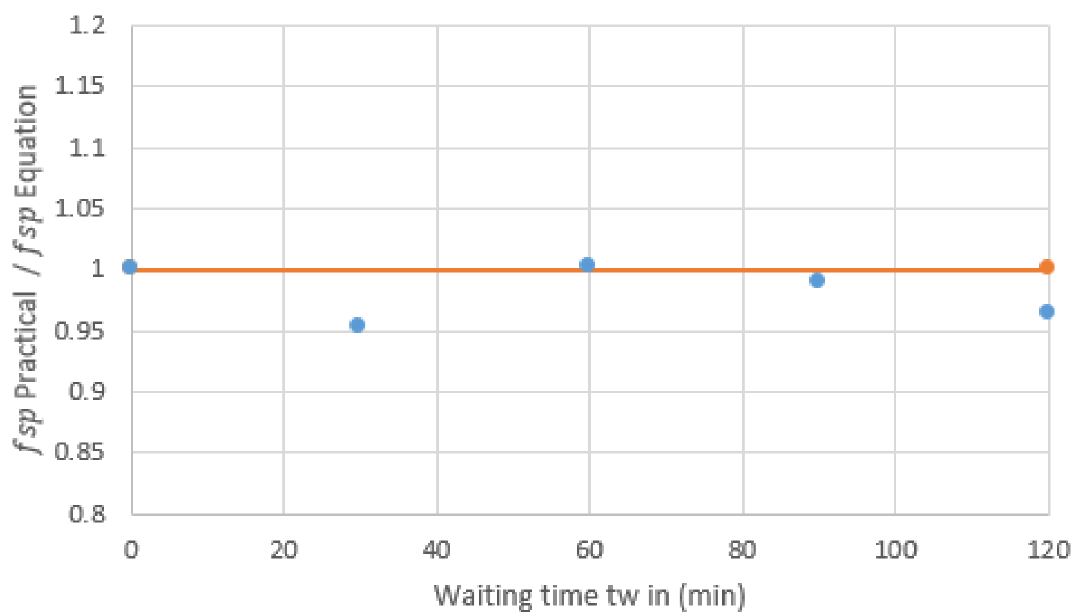

Fig. 5-23 Relative splitting tensile strengths of concrete with re-vibration time (60 sec)

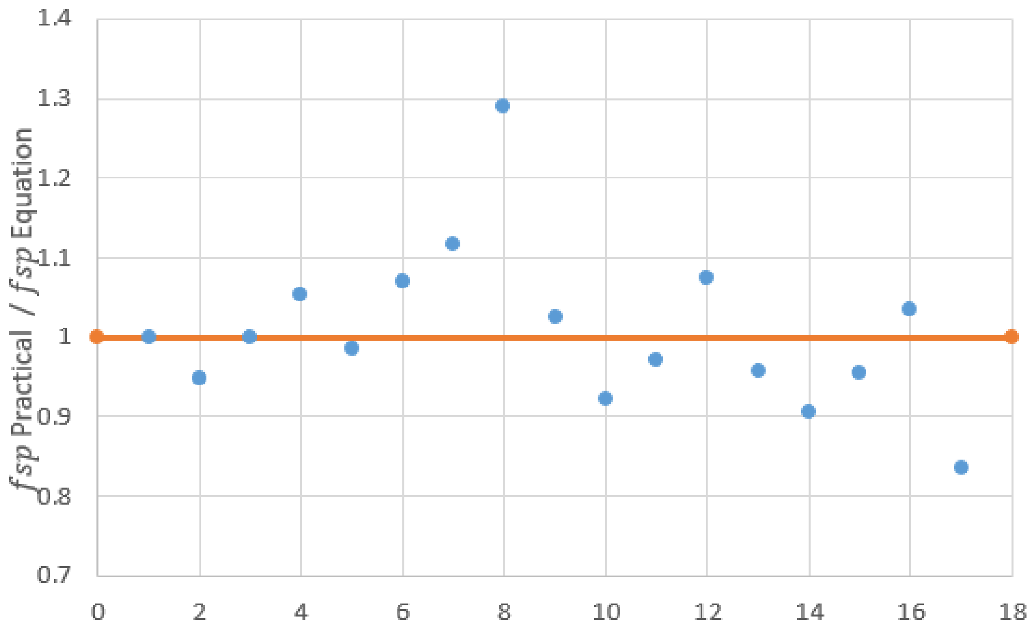

Fig. 5-24 Relative proposed splitting tensile strength for general equation (5-23)

5.6 Proposed concrete equations for first crack load (P_{cr})

A second degree polynomial equation is proposed to show the effect of waiting time (t_w) after the re-vibration of concrete. Initial vibration is (15 sec) and the re-vibration times (t_{rv}) are equal to (15, 30, 45 and 60 sec) for different waiting time (30, 60, 90 and 120 minute), as shown in table 5-6 and figure 5-25 till 5-33.

Table 5-6 Proposed equations for first crack load (P_{cr})

No	t_{rv} (sec)	Proposed Equation	P_{cr} practical	P_{cr} equa-tion	Ratio	R_{Max}	R_{Min}	R_{avg}	σ	Var	r	r^2
1	15	P_{cr}= [-0.0000238 $t_w{}^2$ + 0.00366 t_w + 1] x $P_{cr.c}$ Eq. (5-24)	42	42	1	1.021	0.964	0.994	0.0212	0.0004	0.916	0.839
			45	45.714	0.984							
			49	48	1.021							
			46	47.742	0.964							
			46	46.056	0.999							
2	30	P_{cr}= [-0.0000309 $t_w{}^2$ + 0.00488 t_w + 1] x $P_{cr.c}$ Eq. (5-25)	42	42	1	1.028	0.969	1.000	0.0207	0.0004	0.950	0.902
			47	46.986	1.000							
			51	49.632	1.028							
			48	49.52	0.969							
			48	47.904	1.002							
3	45	P_{cr}= [-0.00005 $t_w{}^2$ + 0.0073 t_w + 1] x $P_{cr.c}$ Eq. (5-26)	42	42	1	1.040	0.950	0.998	0.0325	0.0011	0.930	0.864
			49	49.323	0.993							
			55	52.866	1.040							
			50	52.629	0.950							
			49	48.612	1.008							
4	60	P_{cr}= [-0.0000547 $t_w{}^2$ + 0.00681 t_w + 1] x $P_{cr.c}$ Eq. (5-27)	42	42	1	1.041	0.948	0.988	0.0375	0.0014	0.903	0.815
			46	48.516	0.948							
			53	50.892	1.041							
			47	49.128	0.957							
			43	43.224	0.995							
			42	42	1	1.135	0.905	1.006	0.057	0.0032	0.579	0.335

5	General Equation	$P_{cr} = \left[0.04 * \left(\frac{t_{rv}^{0.56}}{t_w^{0.06}}\right) - 0.00038 * t_{rv} * \left(\frac{t_{rv}^{0.56}}{t_w^{0.06}}\right) + 1 \right] P_{cr.c}$ Eq. (5-28)	45	47.424	0.949							
			47	48.682	0.965							
			49	48.735	1.005							
			46	47.975	0.959							
			49	47.203	1.038							
			51	48.41	1.054							
			55	48.461	1.135							
			53	47.732	1.110							
			46	47.078	0.977							
			48	48.255	0.995							
			50	48.306	1.035							
			47	47.594	0.988							
			46	46.991	0.979							
			48	48.148	0.997							
			49	48.198	1.017							
			43	47.498	0.905							

In this table above the general case is a general equation to predict the concrete where first crack load (P_{cr}) results in the term of both re-vibration times and waiting time using multilinear regression analysis.

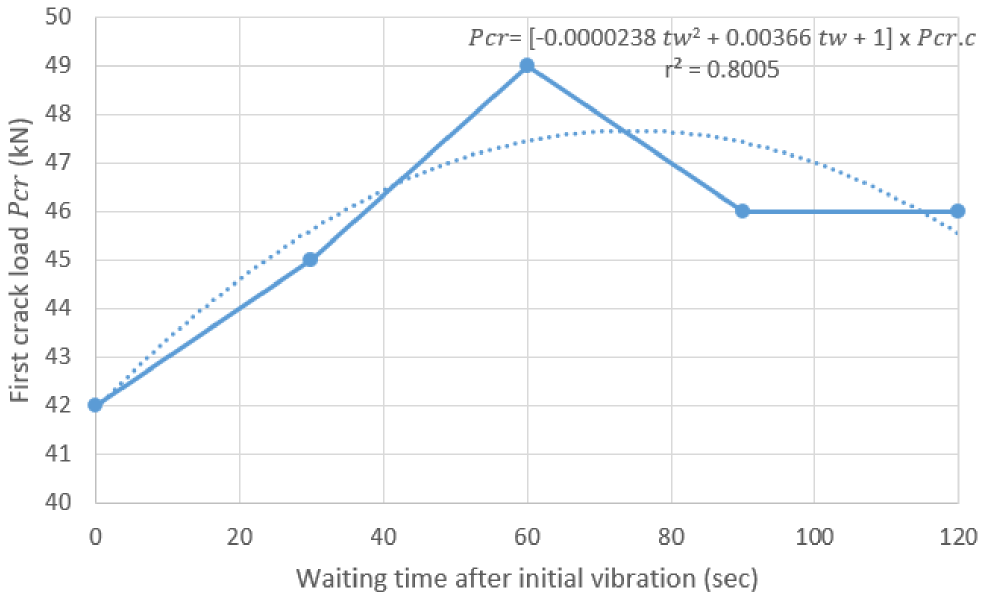

Fig. 5-25 First crack results of the beam with re-vibration time (15 sec) with different waiting time eq. (5-24)

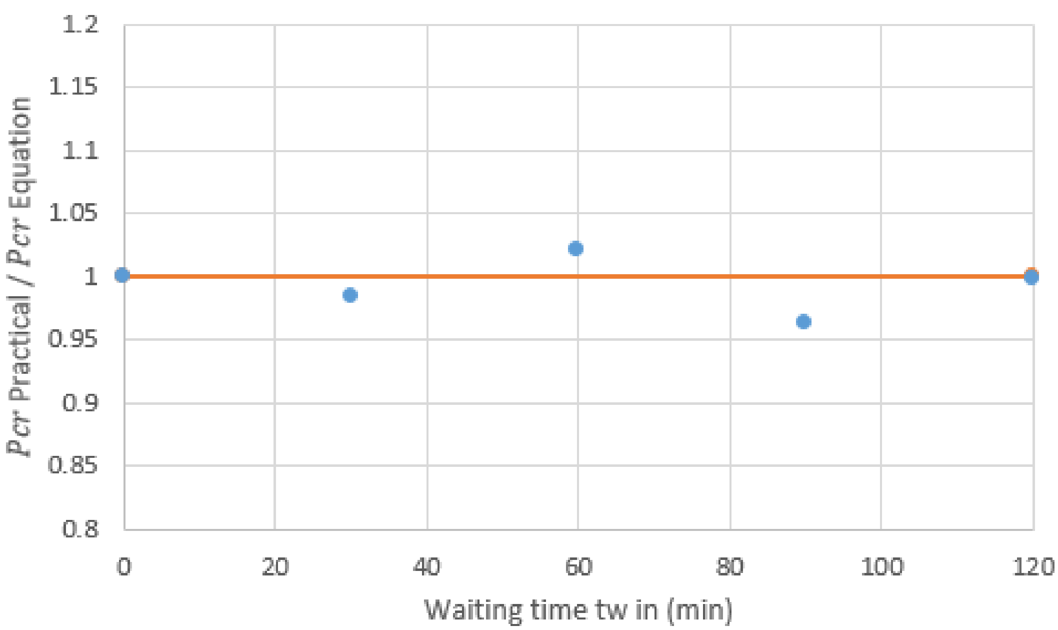

Fig. 5-26 Relative first crack results of the beam with re-vibration time (15 sec)

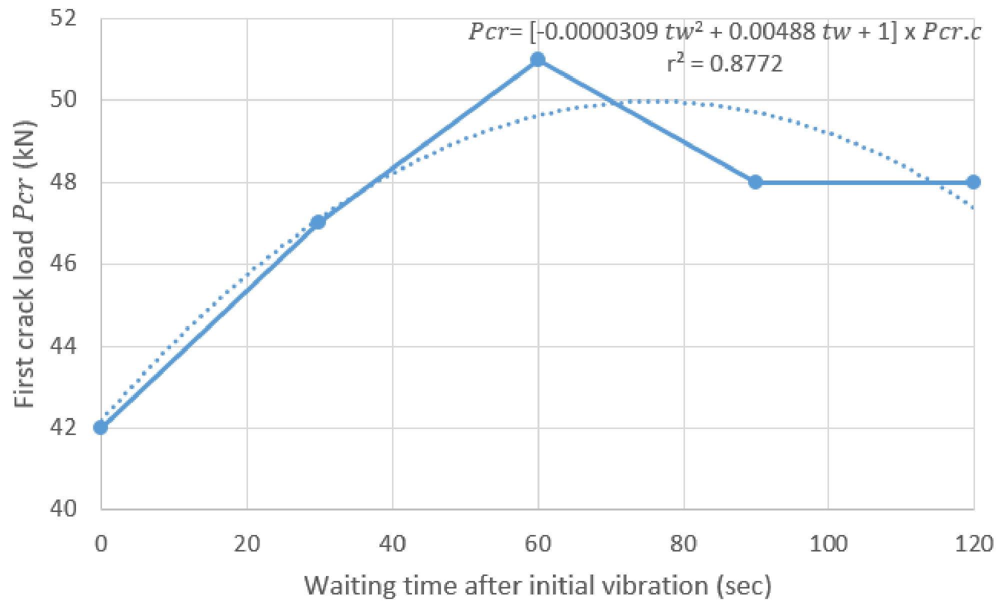

Fig. 5-27 First crack results of beam with re-vibration time (30 sec) with different waiting time eq. (5-25)

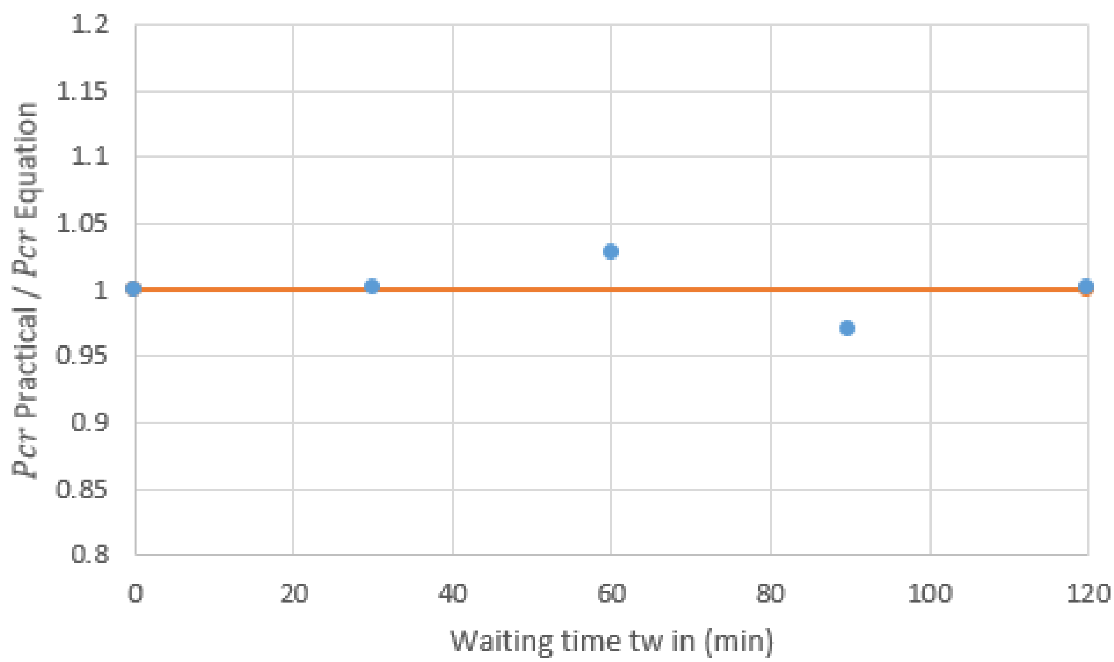

Fig. 5-28 Relative first crack results of beam with re-vibration time (30 sec)

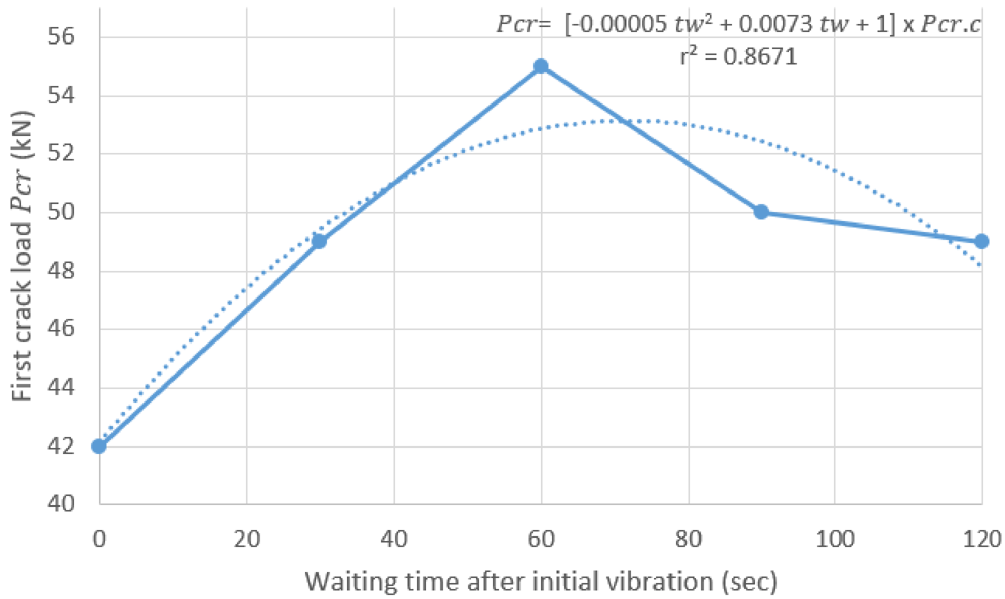

Fig. 5-29 First crack results of beam with re-vibration time (45 sec) with different waiting time eq. (5-26)

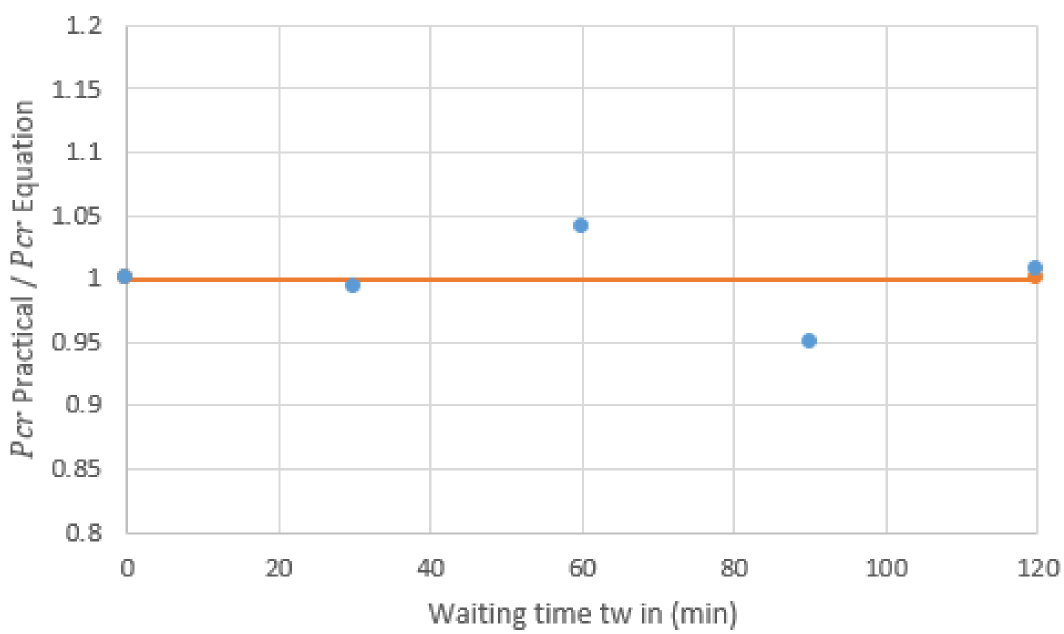

Fig. 5-30 Relative first crack results of beam with re-vibration time (45 sec)

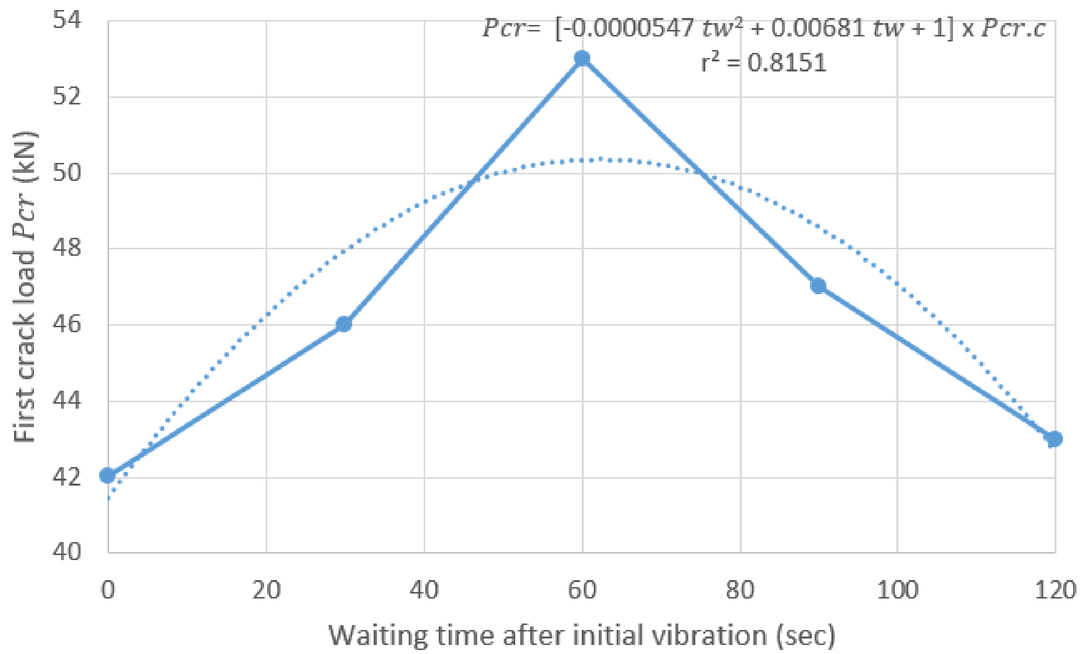

Fig. 5-31 First crack results of beam with re-vibration time (60 sec) with different waiting time eq. (5-27)

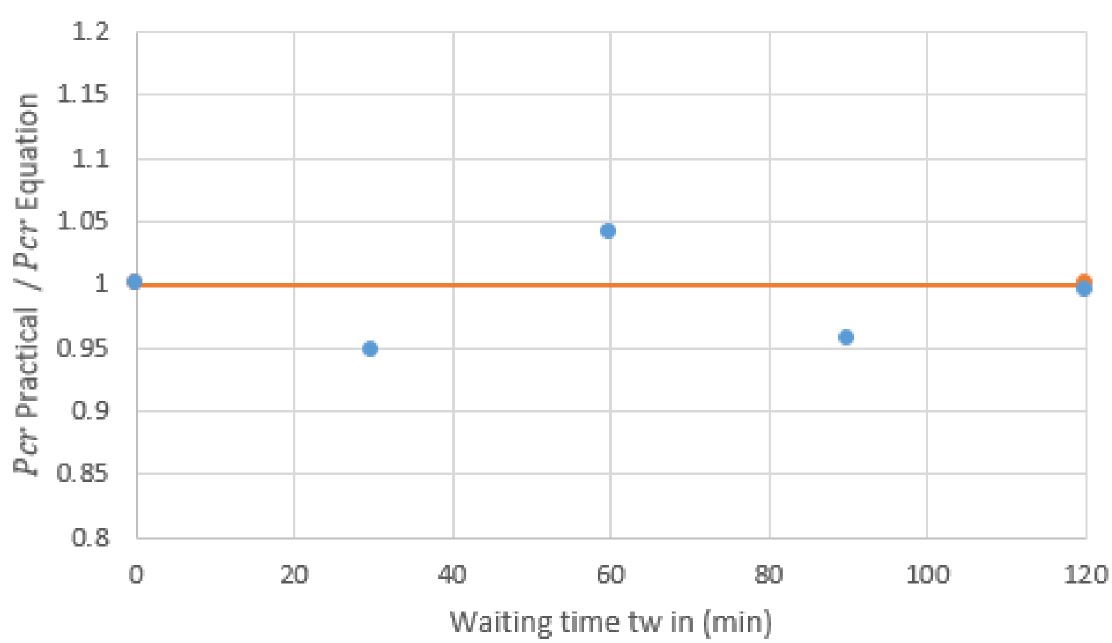

Fig. 5-32 Relative first crack results of beam with re-vibration time (60 sec)

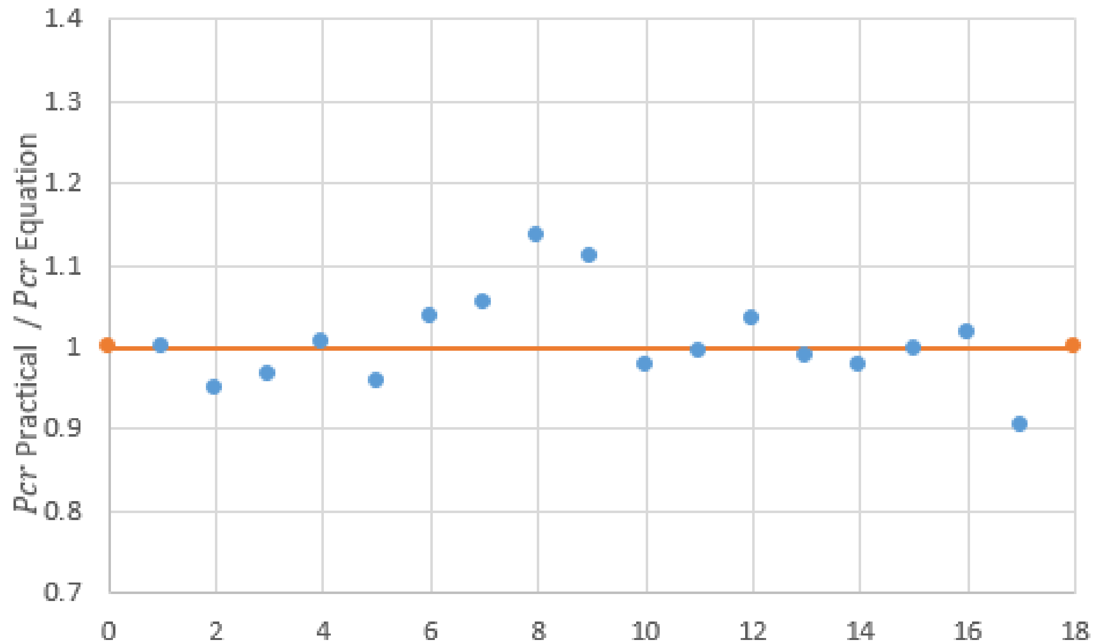

Fig. 5-33 Relative proposed first crack load for general equation (5-28)

5.7 Proposed concrete equations for ultimate load (P_u)

A second degree polynomial equation is proposed to show the effect of waiting time (t_w) after the re-vibration of concrete. Initial vibration is (15 sec) and the re-vibration times (t_{rv}) are equal to (15, 30, 45 and 60 sec) for different waiting time (30, 60, 90 and 120 minute), as shown in table 5-7 and figure 5-34 till 5-42.

Table 5-7 Proposed equations for ultimate load (P_u)

No	t_{rv} (sec)	Proposed Equation	P_u practical	P_u equation	Ratio	R_{Max}	R_{Min}	R_{avg}	σ	Var	r	r^2
1	15	P_u= [-0.0000098 $t_w{}^2$ + 0.00105 t_w + 1] x $P_{u.c}$ Eq. (5-29)	102	102	1	1.020	0.985	1.003	0.0144	0.0002	0.739	0.545
			104	104.328	0.997							
			107	104.856	1.020							
			102	103.584	0.985							
			102	100.512	1.015							
2	30	P_u= [-0.0000245 $t_w{}^2$ + 0.00318 t_w + 1] x $P_{u.c}$ Eq. (5-30)	102	102	1	1.032	0.982	1.009	0.0191	0.0004	0.902	0.814
			113	109.494	1.032							
			114	112.488	1.013							
			109	110.982	0.982							
			107	104.976	1.019							
3	45	P_u= [-0.0000343 $t_w{}^2$ + 0.00443 t_w + 1] x $P_{u.c}$ Eq. (5-31)	102	102	1	1.032	0.979	1.012	0.0223	0.0005	0.925	0.856
			116	112.422	1.032							
			119	116.544	1.021							
			112	114.366	0.979							
			109	105.888	1.029							
4	60	P_u= [-0.0000167 $t_w{}^2$ + 0.00225 t_w + 1] x $P_{u.c}$ Eq. (5-32)	102	102	1	1.012	0.983	0.998	0.0106	0.0001	0.936	0.877
			107	107.355	0.997							
			111	109.65	1.012							
			107	108.885	0.983							
			105	105.06	0.999							
			102	102	1	1.050	0.940	0.977	0.032	0.0010	0.727	0.528

5	General Equation	$P_u = \left(\frac{t_{rv}^{0.1}}{t_w^{0.01}}\right) P_{u(ACI)} [0.89 - 0.0022 * t_{rv} * 0.89]$ Eq. (5-33)	104	109.731	0.948							
			113	113.593	0.995							
			116	114.114	1.017							
			107	113.143	0.946							
			107	108.973	0.982							
			114	112.809	1.011							
			119	113.326	1.050							
			111	112.362	0.988							
			102	108.532	0.940							
			109	112.352	0.970							
			112	112.868	0.992							
			107	111.907	0.956							
			102	108.22	0.943							
			107	112.03	0.955							
			109	112.543	0.969							
			105	111.586	0.941							

In this table above the general case is a general equation to predict the concrete where the ultimate load (P_u) results in the term of both re-vibration times and waiting time using multilinear regression analysis.

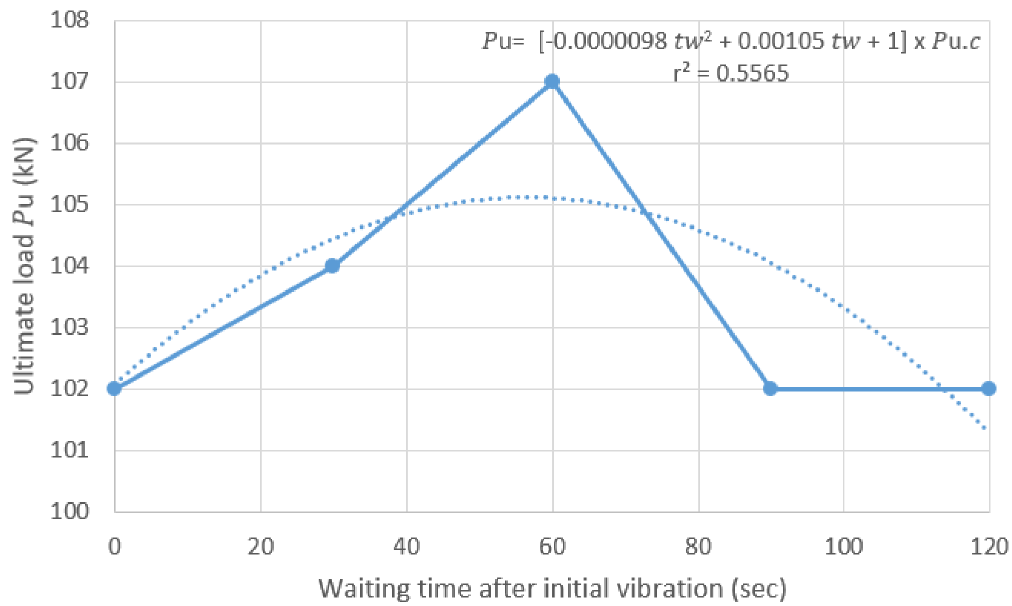

Fig. 5-34 Ultimate load results of the beam with re-vibration time (15 sec) with different waiting time eq. (5-29)

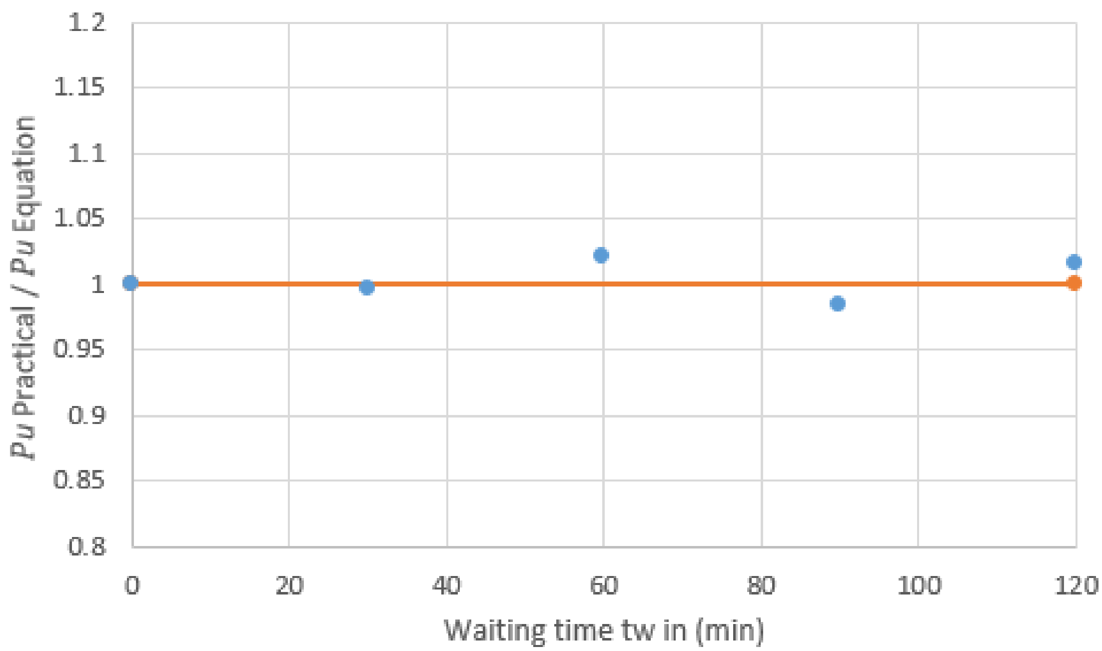

Fig. 5-35 Relative ultimate load results of the beam with re-vibration time (15 sec)

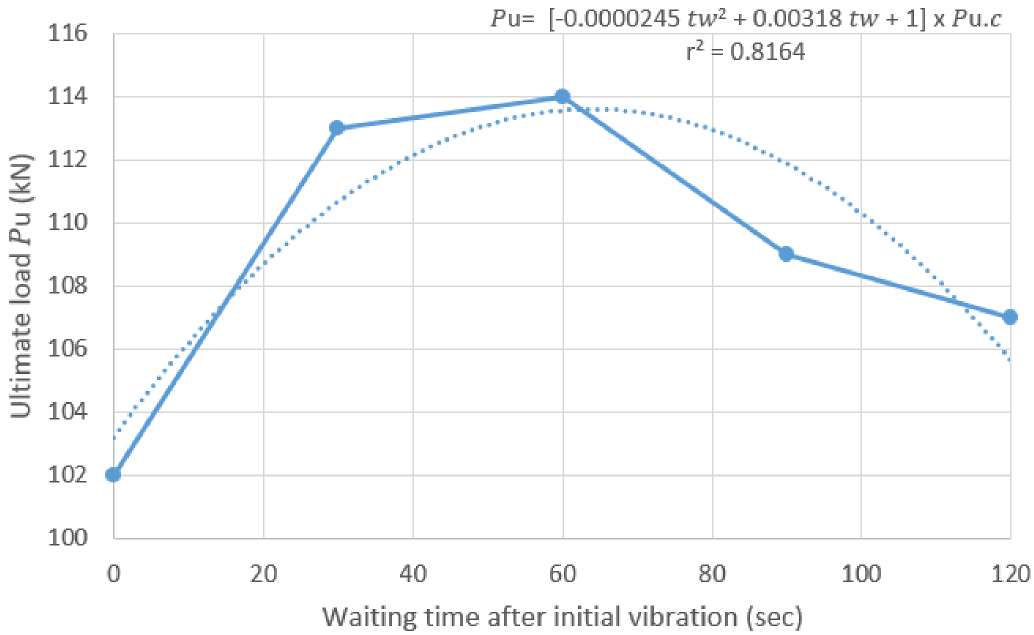

Fig. 5-36 Ultimate load of beam with re-vibration time (30 sec) with different waiting time eq. (5-30)

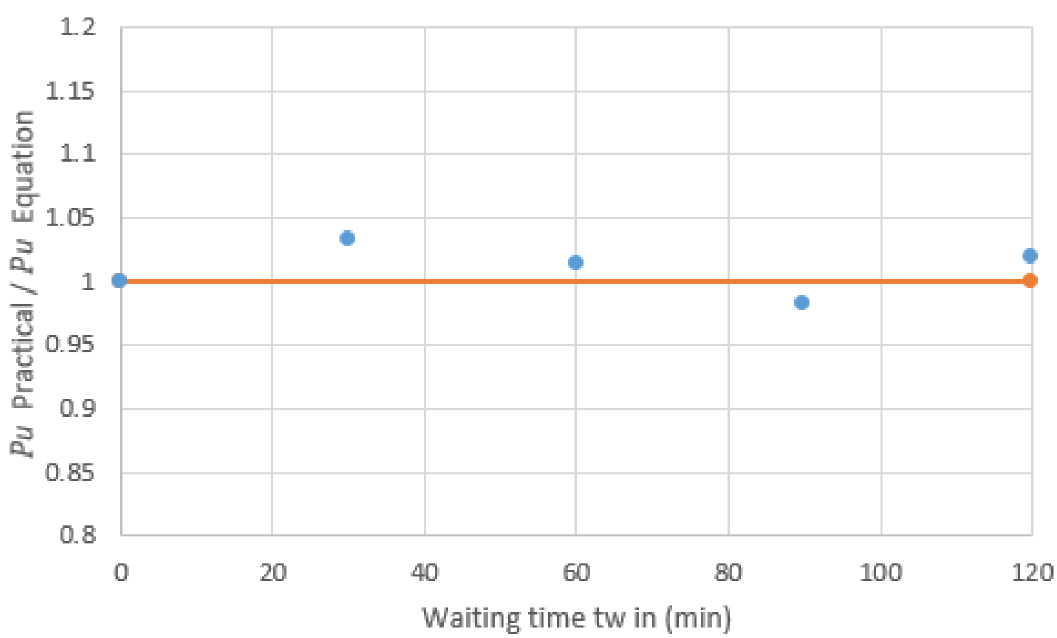

Fig. 5-37 Relative ultimate load of beam with re-vibration time (30 sec)

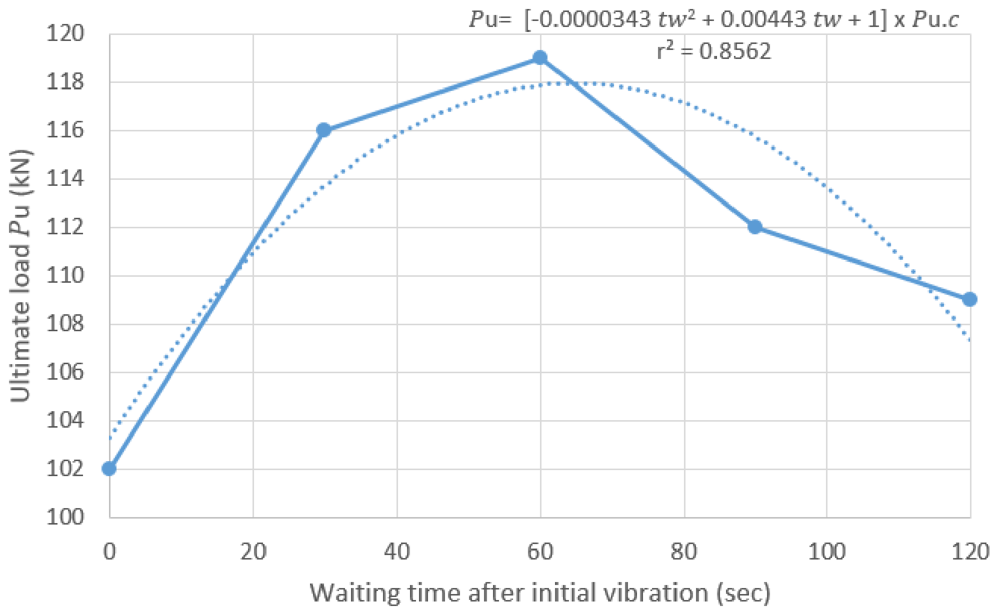

Fig. 5-38 Ultimate load of beam with re-vibration time (45 sec) with different waiting time eq. (5-31)

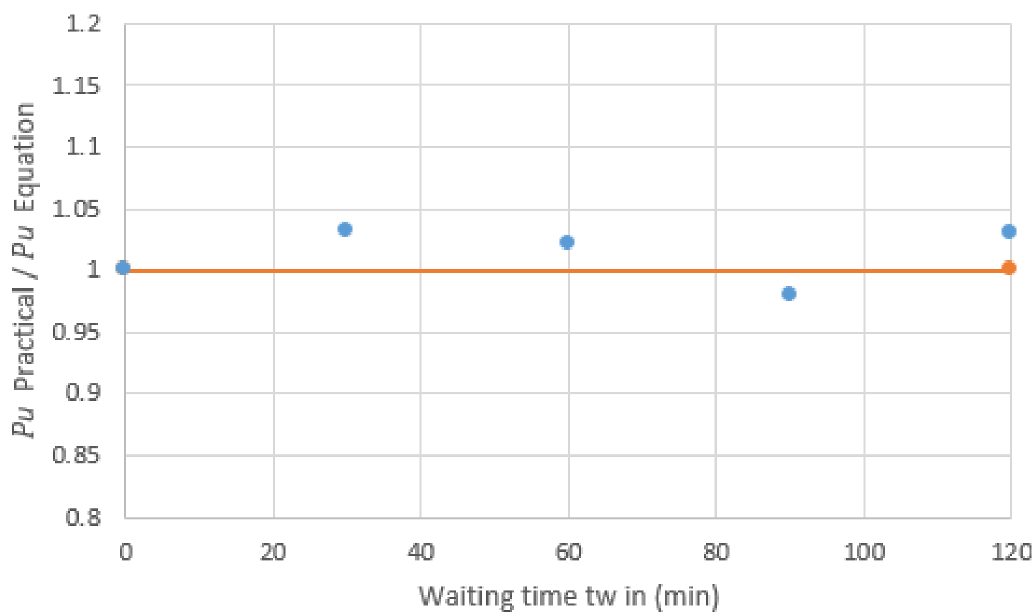

Fig. 5-39 Relative ultimate load of beam with re-vibration time (45 sec)

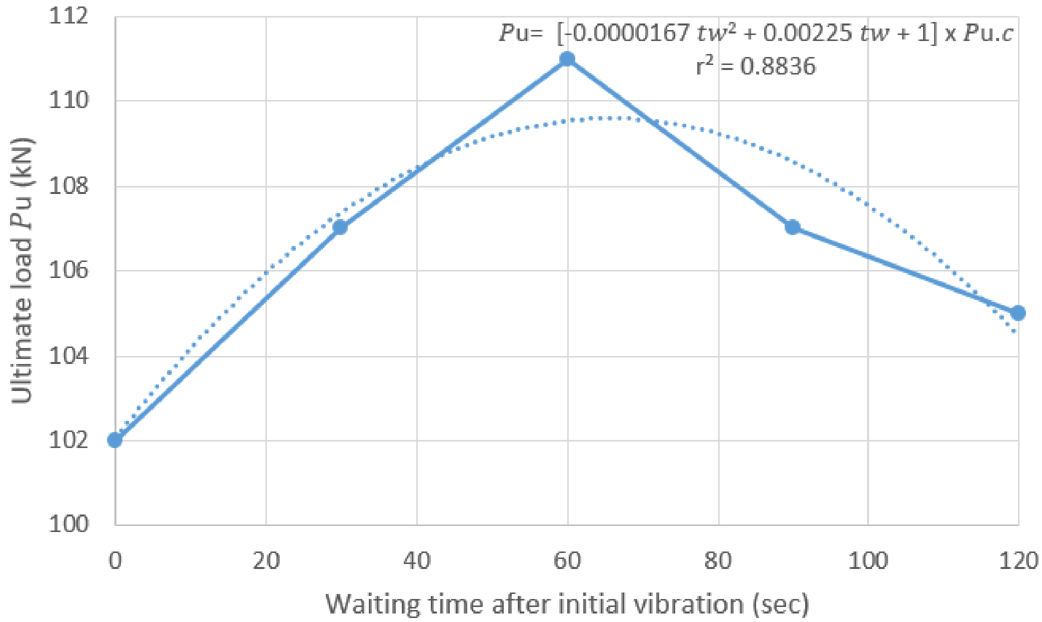

Fig. 5-40 Ultimate load of beam with re-vibration time (60 sec) with different waiting time eq. (5-32)

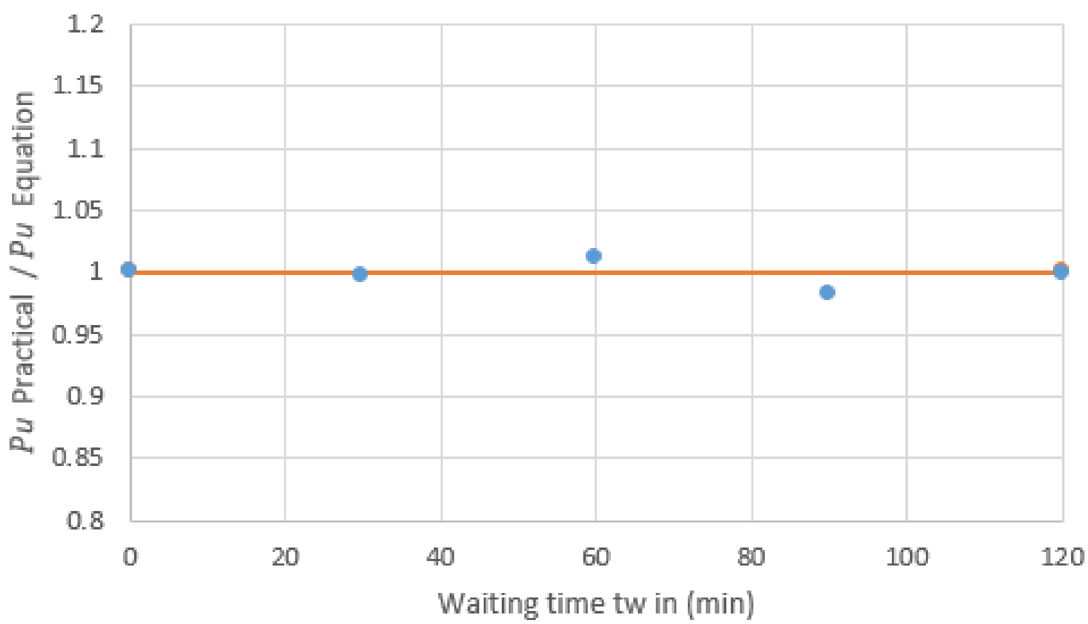

Fig. 5-41 Relative ultimate load of beam with re-vibration time (60 sec)

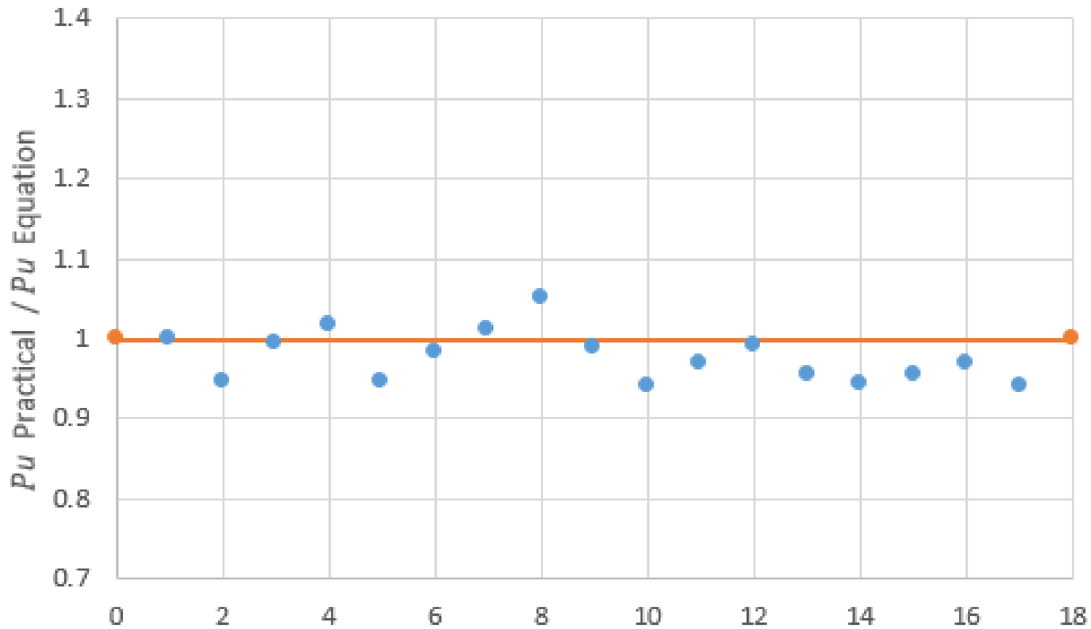

Fig. 5-42 Relative of proposed ultimate load for general equation (5-33)

5.8 Proposed concrete equations for deflection at ultimate load (δ_u)

A second degree polynomial equation is proposed to show the effect of waiting time (t_w) after the re-vibration of concrete. Initial vibration is (15 sec) and the re-vibration times (t_{rv}) are equal to (15, 30, 45 and 60 sec) for different waiting time (30, 60, 90 and 120 minute), as shown in table 5-8 and figure 5-43 till 5-51.

Table 5-8 Proposed equations for deflection at ultimate load (δ_u)

No	t_{rv} (sec)	Proposed Equation	(δ_u) practical	δ_u equation	Ratio	R_{Max}	R_{Min}	R_{avg}	σ	Var	r	r^2
1	15	$\delta_u = [-0.000127\, t_w^2 + 0.0171\, t_w + 1] \times \delta_{u.c}$ Eq. (5-34)	5.5	5.5	1	1.080	0.960	1.029	0.0481	0.0023	0.961	0.924
			8.1	7.705	1.051							
			9.1	8.65	1.052							
			8	8.335	0.960							
			7.3	6.76	1.080							
2	30	$\delta_u = [-0.000163\, t_w^2 + 0.023\, t_w + 1] \times \delta_{u.c}$ Eq. (5-35)	5.5	5.5	1	1.036	0.983	1.010	0.0228	0.0005	0.994	0.987
			8.5	8.509	0.999							
			10.2	9.898	1.031							
			9.5	9.667	0.983							
			8.1	7.816	1.036							
3	45	$\delta_u = [-0.000272\, t_w^2 + 0.0367\, t_w + 1] \times \delta_{u.c}$ Eq. (5-36)	5.5	5.5	1	1.093	0.885	1.005	0.0771	0.0059	0.950	0.903
			10.2	10.207	0.999							
			13.35	12.214	1.093							
			10.2	11.521	0.885							
			8.5	8.128	1.046							
4	60	$\delta_u = [-0.000109\, t_w^2 + 0.015\, t_w + 1] \times \delta_{u.c}$ Eq. (5-37)	5.5	5.5	1	1.042	0.956	1.004	0.0313	0.0010	0.977	0.954
			7.78	7.465	1.042							
			8.5	8.35	1.018							
			7.8	8.155	0.956							
			6.9	6.88	1.003							

			5.5	5.5	1							
			8.1	8.347	0.970							
			8.5	8.488	1.001							
			10.2	8.574	1.190							
			7.78	8.637	0.901							
			9.1	8.174	1.113							
			10.2	8.307	1.228							
			13.35	8.388	1.592							
5	General Equation	$\delta_u = \left[0.58 * \left(\frac{t_{rv}^{0.07}}{t_w^{0.09}}\right) - 0.000181 * \left(\frac{t_{rv}^{0.07}}{t_w^{0.09}}\right) + 1\right] \text{ x } \delta_{u.c}$ Eq. (5-38)	8.5	8.447	1.006	1.592	0.834	1.064	0.179	0.0319	0.510	0.260
			8	8.079	0.990							
			9.5	8.207	1.158							
			10.2	8.285	1.231							
			7.8	8.341	0.935							
			7.3	8.013	0.911							
			8.1	8.138	0.995							
			8.5	8.214	1.035							
			6.9	8.269	0.834							

In this table above the general case is a general equation to predict the concrete where the deflection at ultimate load (δ_u) accurse in the term of both re-vibration times and waiting time using multilinear regression analysis.

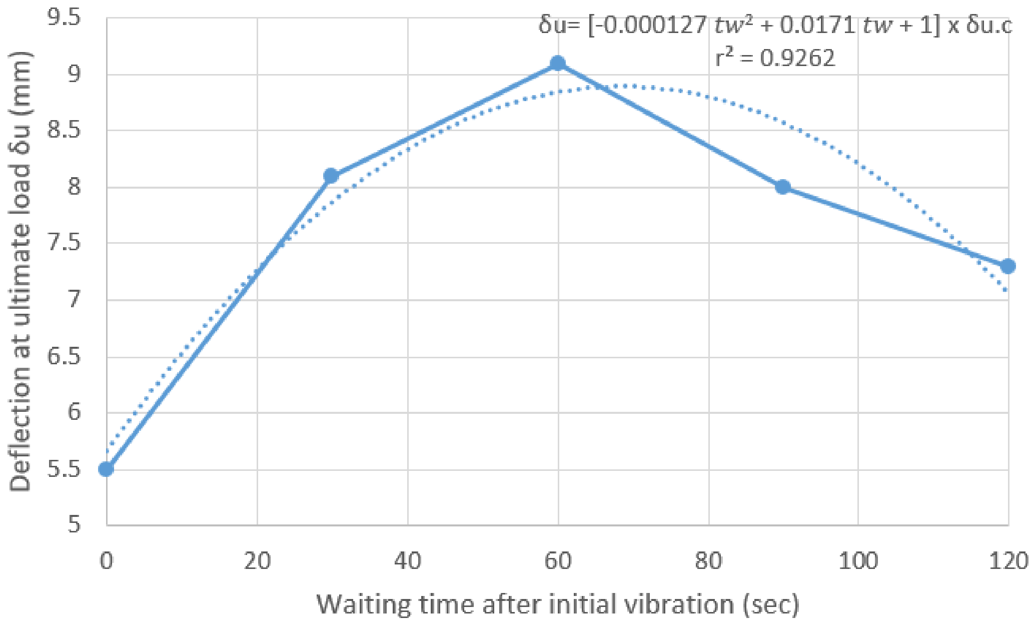

Fig. 5-43 Ultimate load deflection results of the beam with waiting time for re-vibration time length (15 sec.) eq. (5-34)

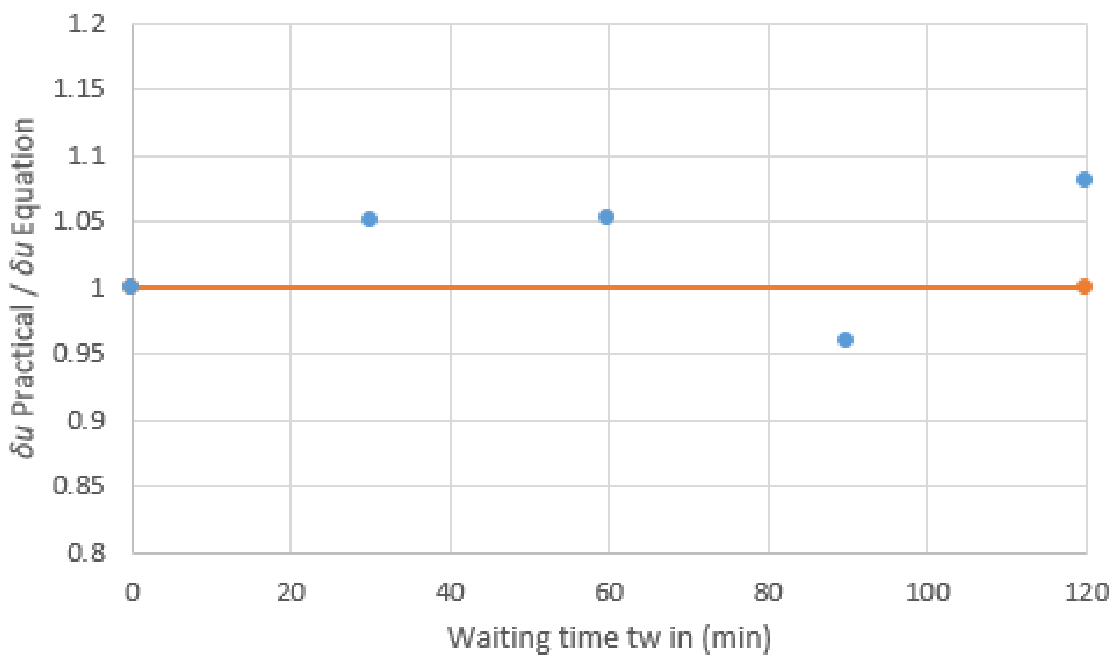

Fig. 5-44 Relative ultimate load deflection results of the beam with waiting time for re-vibration time length (15 sec.)

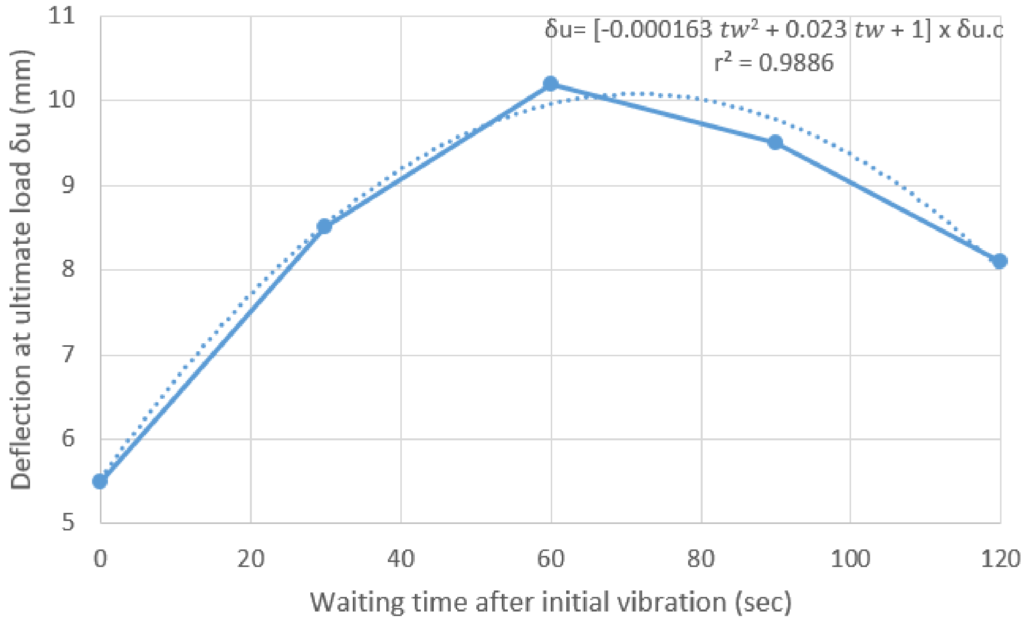

Fig. 5-45 Ultimate load deflection results of the beam with waiting time for re-vibration time length (30 sec.) eq. (5-35)

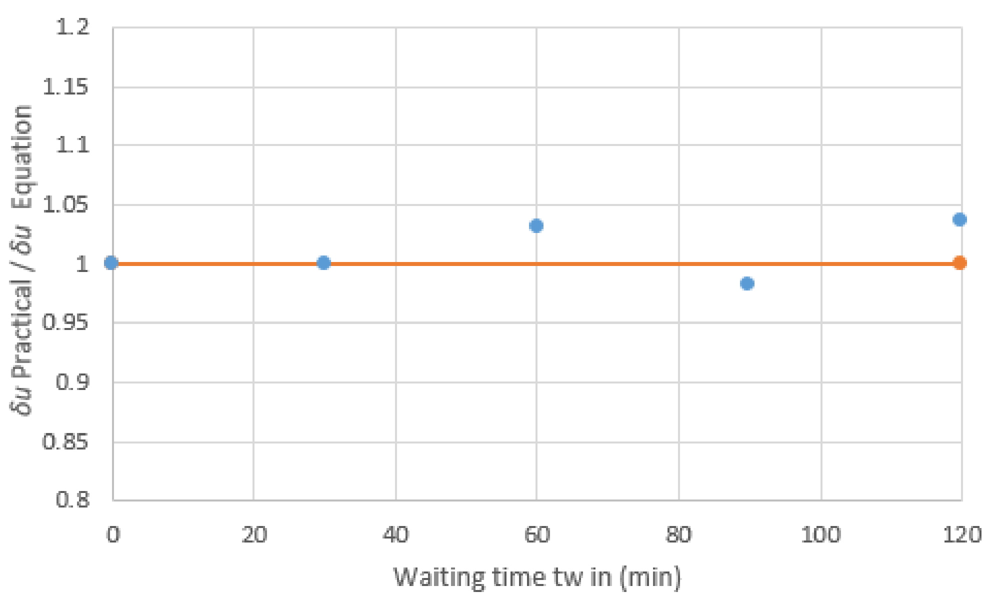

Fig. 5-46 Relative ultimate load deflection results of the beam with waiting time for re-vibration time length (30 sec.)

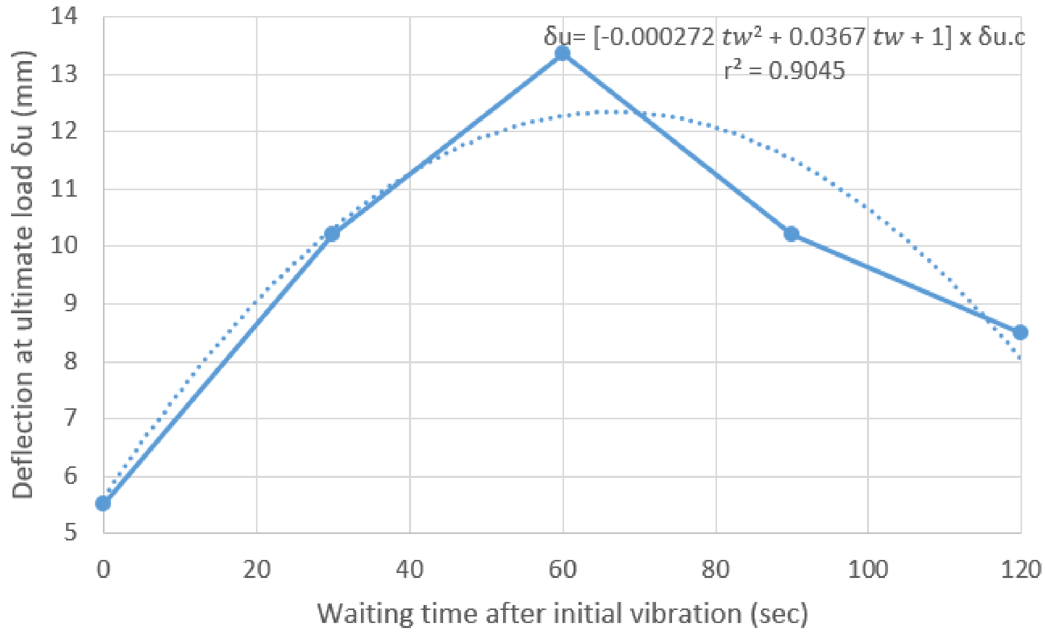

Fig. 5-47 Ultimate load deflection results of the beam with waiting time for re-vibration time length (45 sec.) eq. (5-36)

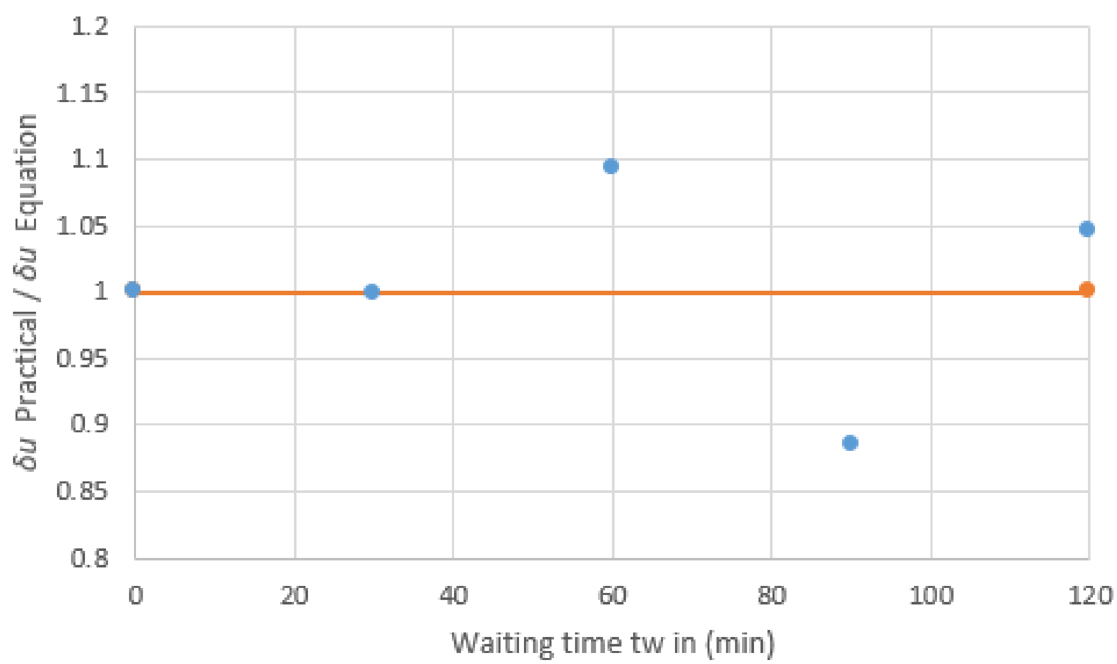

Fig. 5-48 Relative ultimate load deflection results of the beam with waiting time for re-vibration time length (45 sec.)

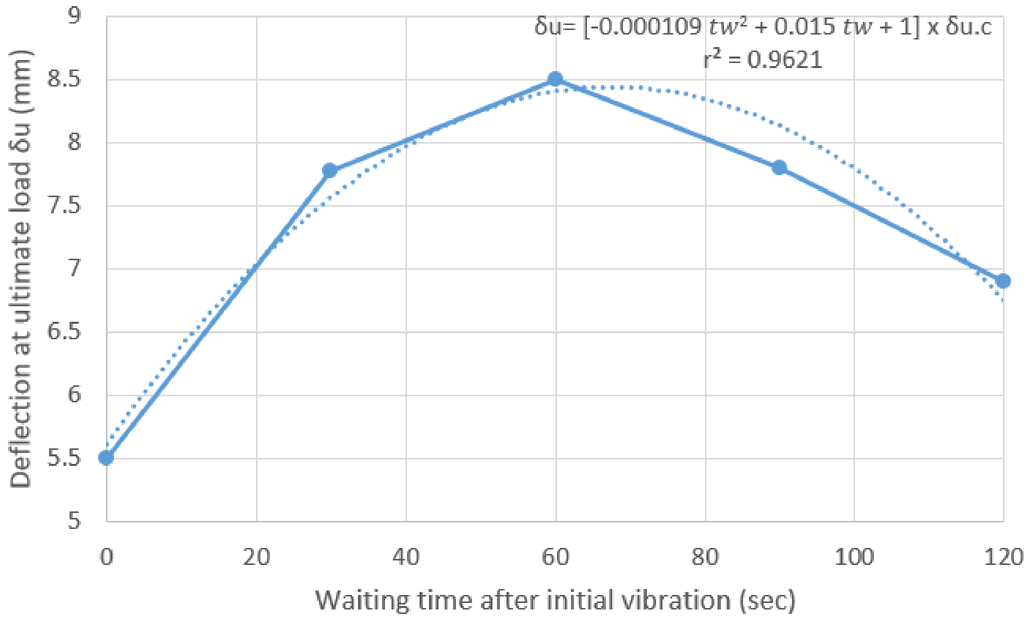

Fig. 5-49 Ultimate load deflection results of the beam with waiting time for re-vibration time length (60 sec.) eq. (5-37)

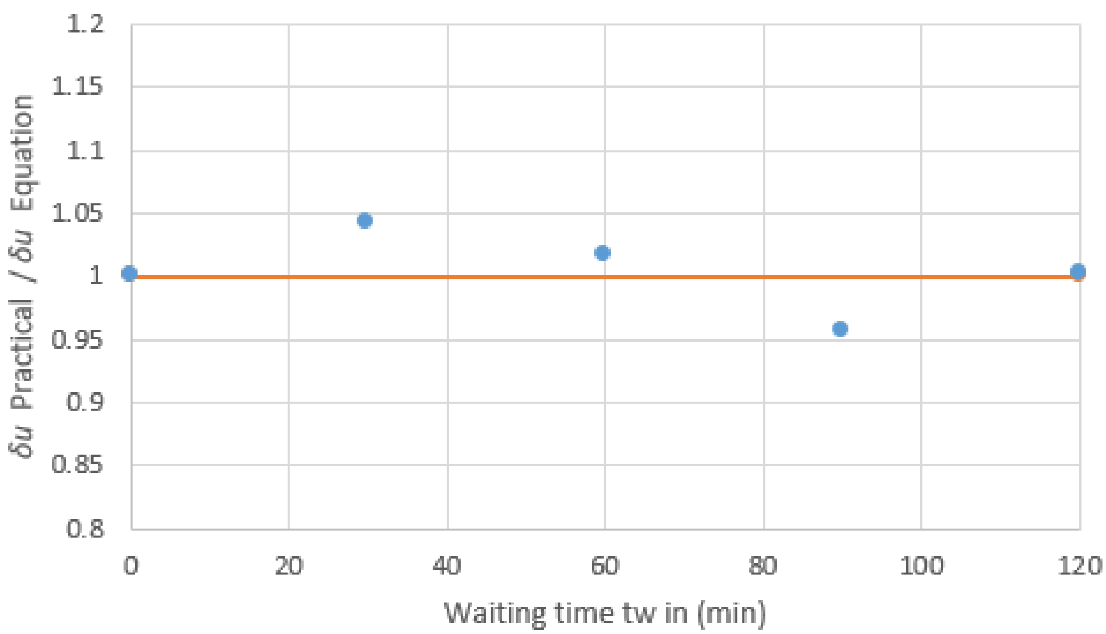

Fig. 5-50 Relative ultimate load deflection results of the beam with waiting time for re-vibration time length (60 sec.)

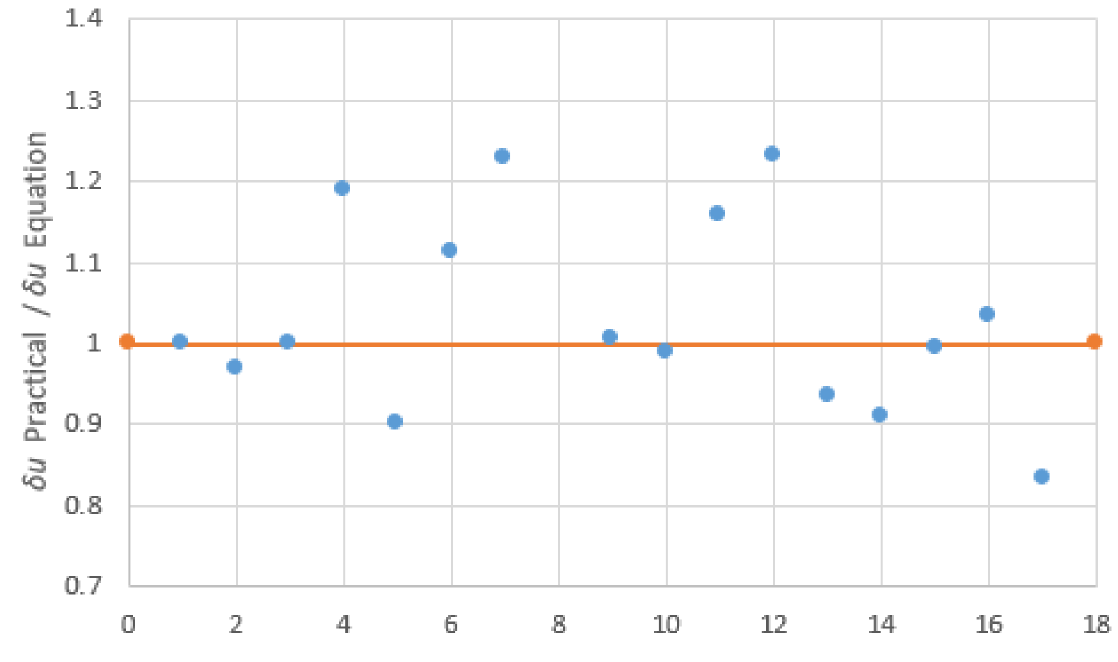

Fig. 5-51 Relative proposed deflection for general equation (5-38)

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

This study focuses on the effect of waiting time before the re-vibration and variable time duration for vibration and re-vibration operation on the structural response of flexural reinforced concrete beams and studying the effect of re-vibration on the mechanical properties of concrete using Ordinary Portland Cement-type 1, with w/c ratio of 0.4 and with more number of waiting time before re-vibration time lag intervals ranging from ½ to 2 hours with different time duration of vibrations ranging from 15 to 60 seconds.

The following conclusions have been summarized from the experimental results obtained from the testing of the beams:

1. The experimental results show that the re-vibration process of the beams after a period of time re-arranged the aggregate particles and eliminates entrapped water which is beneficial to improve the mechanical properties of concrete and increase the failure load (cracking load) of the beams.

2. It is necessary to investigate the effect of the waiting time before the re-vibration and time duration of vibrations on the structural response of flexural reinforced concrete beams to establish and verify the best process to apply this technique. The results show that for the time duration and re-vibration techniques the mechanical properties was increased for the 1st one hour and re-vibrated for 45 seconds time duration. After that for waiting

time 1.5 and 2 hours before re-vibration and re-vibrated for 60 seconds was decreased.

3. In group (A, B, C, D, E and F) were the reinforces beams subjected to applied load during the tests, initial cracks were appeared hairline cracks and distributed spirally across the section along with an amount of small inclined cracks. The first crack appears at the center of the beams. These cracks dispersed throughout both sides of the beams. in group F, which are without transvers reinforcement beam specimens (F1, F2, F3 and F4), the first crack occurred near the ends of the lever arm on both sides of the specimens, it was found that the cracks developed an angle of (45^{o}) with the sides of the beams. In group G, beam specimens (G1, G2, G3, and G4) sudden failure of the beams appeared after the appearance of the initial cracks.

4. The concrete mixtures made with the different re-vibration time lags showed that the maximum compressive strengths of concrete obtained in group A were they vibrated for one time and for 45 seconds without re-vibration was (35.83 MPA), were increased up to 6.92 % and 20.7% when the application of a second vibration was applied after 0.5 and 1 hours then decreased.

5. The maximum tensile strengths of concrete obtained in group A were they vibrated for one time and for 45 seconds without re-vibration was (3.62 MPA), were increased up to 20 % when the application of a second vibration was applied after 1 hour then decreased.

6. Failure load of beams obtained in group A were they vibrated for one time and for 45 seconds without re-vibration was (109 KN), were increased up to 6.4 % and 9 % when the application of a second vibration was applied after 0.5 and 1 hours then decreased.

7. First crack load of beams obtained in group A were they vibrated for one time and for 45 seconds without re-vibration was (45 KN), were increased up to 8 % and 22 % when the application of a second vibration was applied after 0.5 and 1 hours then decreased.

8. The experimental results show that the maximum deflection of the beam was obtained in groups C which was (13.35 mm) were the waiting time for the re-vibration was 1 hour and re-vibrated for 45 seconds.

9. The study found that the strain was also increased with the re-vibration process up to 36.51% when the waiting time for the re-vibration was 1 hour and re-vibrated for 45 seconds then decreased.

10. For the beams of group F that was without transvers reinforcement the re-vibration process also increases the mechanical properties of concrete for compressive strength 10.95%, tensile strength 16.66%, and for failure load increased up to 8.4% when the beams were re-vibrated for 45 seconds then decreased.

11. For the beams of group F, the strain was also increased up to 55.32% when the beams were re-vibrated for 45 seconds then decreased.

12. This study includes a total of 26 proposed equations for beam moment capacity and mechanical properties of concrete as mentioned below:

a- Proposed equation for bending moment (M_o) is :

$$M_o = 0.89\ M_n(1 - 0.0025 * t_{rv})\left(\frac{t_{rv}{}^{0.1}}{t_w{}^{0.01}}\right) \quad (5\text{-}13)$$

b- Proposed equations for concrete compressive strength (f_c') are:

f_c' = [-0.0000486 $t_w{}^2$ + 0.00567 t_w + 1] x $f'_{c.c}$ (5-14) for t_{rv} = 15 sec

f_c' =[-0.0000517 $t_w{}^2$ + 0.0065 t_w + 1] x $f'_{c.c}$ (5-15) for t_{rv} = 30 sec

f_c' = [-0.0000669 $t_w{}^2$ + 0.0085 t_w + 1] x $f'_{c.c}$ (5-16) for t_{rv} = 45 sec

f_c' = [-0.0000395 $t_w{}^2$ + 0.0044 t_w + 1] x $f'_{c.c}$ (5-17) for t_{rv} = 60 sec

$f_c' = \{0.03[\left(\frac{t_{rv}^{0.66}}{t_w^{0.31}}\right) \text{x} (1.9 - 0.025\ * t_{rv}) + 1\} f'_{c.c}$ (5-18) G. eq.

c- Proposed equations for splitting tensile strength (f_{sp}) are:

f_{sp}= [-0.000031 $t_w{}^2$ + 0.00369 t_w + 1] x $f_{sp.c}$ (5-19) for t_{rv} = 15 sec

f_{sp}= [-0.000031 $t_w{}^2$ + 0.00478 t_w + 1] x $f_{sp.c}$ (5-20) for t_{rv} = 30 sec

f_{sp}= [-0.000062 $t_w{}^2$ + 0.008 t_w + 1] $f_{sp.c}$ (5-21) for t_{rv} = 45 sec

f_{sp}= [-0.0000248 $t_w{}^2$ + 0.00186 t_w + 1] $f_{sp.c}$ (5-22) for t_{rv} = 60 sec

$f_{sp} = \left[\ 0.372 * \left(\frac{t_{rv}^{0.45}}{t_w^{0.6}}\right) - 0.0062 * t_{rv} * \left(\frac{t_{rv}^{0.45}}{t_w^{0.6}}\right) + \ 1\right] f_{sp.c}$ (5-23) G. eq.

d- Proposed equations for first crack load (P_{cr}) are:

P_{cr}= [-0.0000238 $t_w{}^2$ + 0.00366 t_w + 1] x $P_{cr.c}$ (5-24) for t_{rv} = 15 sec

P_{cr}= [-0.0000309 $t_w{}^2$ + 0.00488 t_w + 1] x $P_{cr.c}$ (5-25) for t_{rv} = 30 sec

P_{cr}= [-0.00005 $t_w{}^2$ + 0.0073 t_w + 1] x $P_{cr.c}$ (5-26) for t_{rv} = 45 sec

P_{cr}= [-0.0000547 $t_w{}^2$ + 0.00681 t_w + 1] x $P_{cr.c}$ (5-27) for t_{rv} = 60 sec

$$P_{cr} = \left[\, 0.04 * \left(\frac{t_{rv}^{0.56}}{t_w^{0.06}}\right) - 0.00038 \, * t_{rv} * \left(\frac{t_{rv}^{0.56}}{t_w^{0.06}}\right) + 1\right] P_{cr.c} \quad \text{(5-28) G. eq.}$$

e- Proposed equations for ultimate load (P_u) are:

P_u= [-0.0000098 $t_w{}^2$ + 0.00105 t_w + 1] x $P_{u.c}$ (5-29) for t_{rv} = 15 sec

P_u= [-0.0000245 $t_w{}^2$ + 0.00318 t_w + 1] x $P_{u.c}$ (5-30) for t_{rv} = 30 sec

P_u= [-0.0000343 $t_w{}^2$ + 0.00443 t_w + 1] x $P_{u.c}$ (5-31) for t_{rv} = 45 sec

P_u= [-0.0000167 $t_w{}^2$ + 0.00225 t_w + 1] x $P_{u.c}$ (5-32) for t_{rv} = 60 sec

$$P_u = \left(\frac{t_{rv}{}^{0.1}}{t_w{}^{0.01}}\right) P_{u(ACI)}[0.89 - 0.0022 * t_{rv} * 0.89] \quad \text{(5-33) G. eq.}$$

f- Proposed equations for deflection at ultimate load (δ_u) are:

δ_u= [-0.000127 $t_w{}^2$ + 0.0171 t_w + 1] x $\delta_{u.c}$ (5-34) for t_{rv} = 15 sec

δ_u= [-0.000163 $t_w{}^2$ + 0.023 t_w + 1] x $\delta_{u.c}$ (5-35) for t_{rv} = 30 sec

δ_u= [-0.000272 $t_w{}^2$ + 0.0367 t_w + 1] x $\delta_{u.c}$ (5-36) for t_{rv} = 45 sec

δ_u= [-0.000109 $t_w{}^2$ + 0.015 t_w + 1] x $\delta_{u.c}$ (5-37) for t_{rv} = 15 sec

$$\delta_u = \left[0.58 * \left(\frac{t_{rv}^{0.07}}{t_w^{0.09}}\right) - 0.000181 * \left(\frac{t_{rv}^{0.07}}{t_w^{0.09}}\right) + 1\right] \text{ x } \delta_{u.c} \quad \text{(5-38) G. eq.}$$

6.2 Suggestion for the Engineering Designers

As a suggestion for the designer engineers, during this study, it has been found that, that the mechanical properties of concrete (compressive strength, tensile strength, flexural strength, and modulus of elasticity) failure load and strain of concrete with various time duration and re-vibration techniques was increased for the 1st one hour and re-vibrated for 45 seconds time duration. So using the re-vibration time of 45 seconds is beneficial for this purpose.

6.3 Recommendations and Future Work

1. Study the effect of re-vibration process on the behavior of high strength concrete beams.
2. Study the effect of re-vibration process on the beams with hollow beams.
3. Study the effect of re-vibration process on the behavior of beams with stir-rups only in the study area.
4. Study the different vibration techniques and study the concrete effect.
5. Study the effect of different concrete grades.
6. Study the effect of different water cement ratio on the re-vibration process.

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Appendix A

Shear calculation using equation 5-1:

L=1300mm d=220mm f'_c =32.873MPa f_y=628MPa $S1 = 160$mm

a= $\frac{1300-160}{2}$ = 570 mm so $\frac{a}{d}$ = 2.59

$$V_c = \frac{\lambda}{6}\sqrt{f'_c} b_w d$$

$V_c = \frac{1}{6}\sqrt{32.873}\ \frac{125*220}{1000} = 26.28$ KN so $P_{cr} = 52.56$ KN

$A_v = 2 * \frac{\pi}{4}(10)^2 = 157.08\ mm^2$

$V_s = \frac{A_v\ fy\ d}{S}$ / 1000

$$V_s = (\frac{157.08 * 628 * 220}{100})/1000 = 217.02 \text{ KN}$$

$V_u = V_c + V_s = 62.28 + 217.02 = 243.3$ KN

$P_u = 2 * V_u = 486.6$ KN

Shear calculation using equation 5-2:

$A_s = 2\frac{\pi}{4}(12)^2 = 226.2\ mm^2$

$\rho = \frac{A_s}{b_w d} = \frac{226.2}{125*220} = 0.008225$

$V_c = \frac{2}{3}\lambda\rho^{\frac{1}{3}}\sqrt{f'_c} b_w d = \frac{2}{3}\ 0.008225^{\frac{1}{3}}\sqrt{32.873}\ (125 * 220)/1000 = 21.218$ KN

R=$\frac{21}{21.218}$=0.98

Flexural calculation

$A_v = 2\ \frac{\pi}{4}\ 10^2 = 157.08\ mm^2$

$A_s = 2\ \frac{\pi}{4}\ 12^2 = 226.2\ mm^2$

$\rho = \frac{A_s}{bd} = \frac{226.2}{125*220} = 0.008225$

$M_n = 0.008225 * 125 * 220^2 * 628\left(1 - 0.59 * 0.008225\frac{628}{32.273}\right)/10^6$=28.3 Kn.m

$\frac{P_n * a}{2}$ = 28.3 KN.m

$P_n = \frac{2\ M_n}{a} = \frac{2*(28.353)}{0.57}$ = 99.484 KN …… (Control)

$y' = \frac{bh(\frac{h}{2})+(n-1)As\ d+(n-1)As'd\prime}{bh+(n-1)As+(n-1)As\prime}$

Where:

$n = \frac{E_s}{E_c} = \frac{200000}{4730\sqrt{32.873}}$ = 7.3748 so n-1 = 6.3748

Ig = Moment of inertia of the transformed section

A_s = 226.2 mm^2

A_s' = 157.08 mm^2

$$y' = \frac{125 * 250(\frac{250}{2}) + 6.3748(226.2)(220) + 6.3748(157.08)(30)}{125 * 250 + 6.3748(226.2) + 6.3748(157.08)}$$

y' = 126.242 mm ; $y_t = h - y'$ = 123.758 mm

$$I_g = \frac{b_w h^3}{12} + b_w h\,(y' - \frac{h}{2})^2 + (n-1)A_s\,(d - y')^2 + \left(n' - 1\right)A_s'(y' - d')^2$$

$$I_g = \frac{125 * 250^3}{12} + 125 * 250\ (126.242 - 125)^2 + 6.3748$$
$$* (226.2)(220 - 126.242)^2 + 6.3748 * (157.08)(126.242 - 30)^2$$

$I_g = 22650120.25\ mm^4$

$M_{cr} = \frac{f_r\, I_g}{y_t}$

Where: f_r = modulus of rupture of the concrete = $0.625\sqrt{f_c'}$ (MPa)

$y_t = h - y'$ distance from the neutral axes to the tension face (mm)

$M_{cr} = \frac{3.583*22650120.25}{123.758}$ = 0.6558 KN.m

$M_{cr} = \frac{P_{cr}*a}{2}$ so $P_{cr} = \frac{2*M_{cr}}{a} = \frac{2\ (0.6558)}{0.57}$ = 2.301 KN

Thus for beam A1; first cracking load = 2.301 KN = P_{c1}

Diagonal shear crack = 52.56 KN = P_{c2}

$P_{\text{cr exp.}}$= 42 MPa ;

$\frac{P_{\text{cr exp.}}}{P_{\text{cr cal.}}} = \frac{42}{52.56} = 0.8$

EAJSE
Eurasian Journal of Science and Engineering

MANUSCRIPT ACCEPTANCE LETTER

Date: May 23, 2023

Dear **Ali Dlshad Nooraldeen and Ayad Zaki Saber**

I am pleased to inform you that your paper entitled "**Effect Of Waiting Time Before Re-Vibration On Flexural Behavior Of Reinforced Concrete Beam**" has been accepted for publication in EAJSE and will be published in the June issue of 2023.

If you have any questions, please do not hesitate to contact us.
Best Regards,

Dr. Orhan Tug
Managing Editor

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لێکۆڵینەوە کرداریەکە پێک هاتبوو لە دروست کردن و پشکنینی (28) ڕایەڵی لاکێشەیی کە دابەش کرابون بەسەر حەوت کۆمەلە کەبرتین (A, B, C, D, E, F, G) هەر کۆمەلەیەك چوار نمونە لەخۆدەگرێت بە ئەندازەی 125 ملم پانی و250 ملم بەرزی و 1500 ملم درێژی، هەموو رایەڵەکان شیشبەندیان هەیە بە ئاراستەی درێژی رایەڵەکان (2Φ10) ملم لەسەرەوە لەگەڵ (2Φ12) ملم لەخواروە لەگەڵ بەکارهێنانی (Φ10) ملم شیشبەندی سوران (قەفیز) بۆ هەموو رایەڵەکان جگە لە کۆمەلەی (F) کە بەبێ بەهێزکردنی شیشبەندی سوران (قەفیز) لەگەڵ کۆمەلەی (G) کە بەبێ بەهێزکردنی شیشبەندی درێژی و سوران (قەفیز).

سەرەڕای ئەوەش، 144 سلندەر بە ئەندازەی 300 ملم لە بەرزی و 150 ملم لە تیرەدا بەکارهێنران بۆ هەڵسەنگاندنی هێزی پەستان و کێشانی کۆنکرێتەکە دوای 56 ڕۆژ، ئەم نمونانە شیکرانەوە لە هەوڵێکدا بۆ لێکۆڵینەوە لە کاریگەرییەکانی لەرزین، و کاریگەری دواکەوتنی دووبارە لەرزین و ماوەی لەرزین لەسەر گەشەکردنی هێزی کۆنکرێت.

ئەنجامەکان دەریانخستووە کە زۆرترین تایبەتمەندییە میکانیکییەکانی کۆنکرێت، باری کۆتایی، باری درز یەکەم و لێشێوان لە خاڵی ناوەڕاستدا لەگەڵ ماوەی کات و تەکنیکەکانی لەرزین بۆ ماوەی چاوەڕوانی کاتژمێری یەکەم بەدەست هێنرا کە بۆ ماوەی 45 چرکە لەرزیندرایەوە. دوای ئەوە بۆ ماوەی چاوەڕوانی 1.5 و 2 کاتژمێر و دووبارە لەرزین بۆ ماوەی 60 چرکە کەم بووەوە.

پوختە :
دووبارە لەرزین کە بریتییە لە پرۆسەی دووبارەکردنەوەی کارکردنی لەرزین لە کۆنکرێتی تازە دوای ماوەیەک، ڕەنگە بەسوود بێت بۆ بەرزکردنەوەی تایبەتمەندییە میکانیکییەکانی کۆنکرێت (هێزی پاڵەپەستۆ، هێزی کێشکردن، هێزی چرچبوون، و مۆدیولی لاستیکی)، هەروەها بۆ بەدەستهێنانی زۆرترین باری بەکارهێنراو، یەکەم باری درز، زۆرترین لادان لە ناوەڕاست، ڕەقبوون و پەستان، بە تایبەتی کاتێک چینە یەک لە دوای یەکەکانی کۆنکرێتی تازە دانران و چینە سەرەوەی کۆنکرێتی تازە بەشێکی ڕەق بووەوە. دوای ماوەیەک، تەنۆلکەکانی کۆکراوە بە پرۆسەی لەرزین ڕێکدەخرێنەوە، و هەر ئاوێکی گیری خواردووە لادەبرێت، کە بە ئەگەرێکی زۆرەوە هێزی پەستان و کێشانی کۆنکرێتەکە بەرز دەکاتەوە. بەکارهێنانی دووبارە لەرزین دەتوانێت یارمەتیدەر بێت بۆ لابردنی درزی بچووکبوونەوەی پلاستیکی بۆ کۆنکرێتی بەرکەوتوو. بڕی ئەو کاتەی کە دووبارە لەرزینەوەی بۆدەکرێت کاریگەرییەکی گەورەی هەیە.
بەکارهێنانی تەکنیکەکانی دووبارە لەرزین لە دروستکردنی ئەندامانی پێکهاتەییدا چاوەڕوان دەکرێت تایبەتمەندییە پێکهاتەییەکانی رایەڵەکان باشتر بکات و درزەکان کەمتر بکات. کاریگەری کاتی چاوەڕوانی پێش دووبارە لەرزین دەبێت لێکۆڵینەوەی لەسەر بکرێت و ماوەی کاتی لەرزینەکان لەسەر وەڵامی پێکهاتەی رایەڵی کۆنکرێتی بەهێزکراوبۆ دامەزراندنی و پشتڕاستکردنەوەی باشترین پرۆسەی بەکارهێنانی ئەم تەکنیکە.
ئامانج لەم لێکۆڵینەوەیە پشکنینە کاریگەری کاتی چاوەڕوانی پێش دووبارە لەرزین. و ماوەی کاتیی گۆڕاو بۆ کارکردنی لەرزین و دووبارە لەرزین لە وەڵامی پێکهاتەیی بۆ ڕایەڵی کۆنکرێتی شیشدار, بەکارهێنانی چیمەنتۆی ئاسایی پۆرت-لاند-جۆری 1، لەگەڵ ڕێژەی ئاوی0.4 w/c ، و دواکەوتنی کاتی دووبارە لەرزین کە لە نێوان نیو کاتژمێرەوە تا دوو کاتژمێر بۆ هەڵسەنگاندنی کاریگەری دووبارە لەرزین لەسەر تایبەتمەندییە میکانیکییەکانی کۆنکرێت بە ماوەی جیاوازی لەرزینەکان کە لە نێوان 15 بۆ 60 چرکەدایە.

کاریگەری کاتی چاوەڕوانی پێش دووبارە لەرینەوە لەسەر ڕەفتاری فلێکسوراڵ

لە ڕایەڵی کۆنکریتی شیشدار

نامەیەکە

پێشکەشی ئەنجومەنی کۆلێژی تەکنیکی ئەندازیاری کراوە لەزانکۆی پۆلیتەکنیکی هەولێر

وەکو بەشێك لە پێداویستیەکانی بەدەست هێنانی ماستەر لە (ئەندازیاری شارستانی)

لەلایەن

علی دلشاد نورالدین

بەکالۆریۆس لە ئەندازیاری شارستانی - زانکۆی پۆلیتەکنیکی هەولێر (2015)

بەسەرپەرشتی

پ. د. ئەیاد زکی صابر أغا

کوردستان - هەولێر

2023 - گەلاڕێزان