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Estimation of the characteristic in-situ compressive strength class of concrete structures: A case study of the Skuru bridge

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Estimation of the characteristic in-situ compressive strength class of concrete structures: A case study of the Skuru bridge

A thesis project owned by COWI

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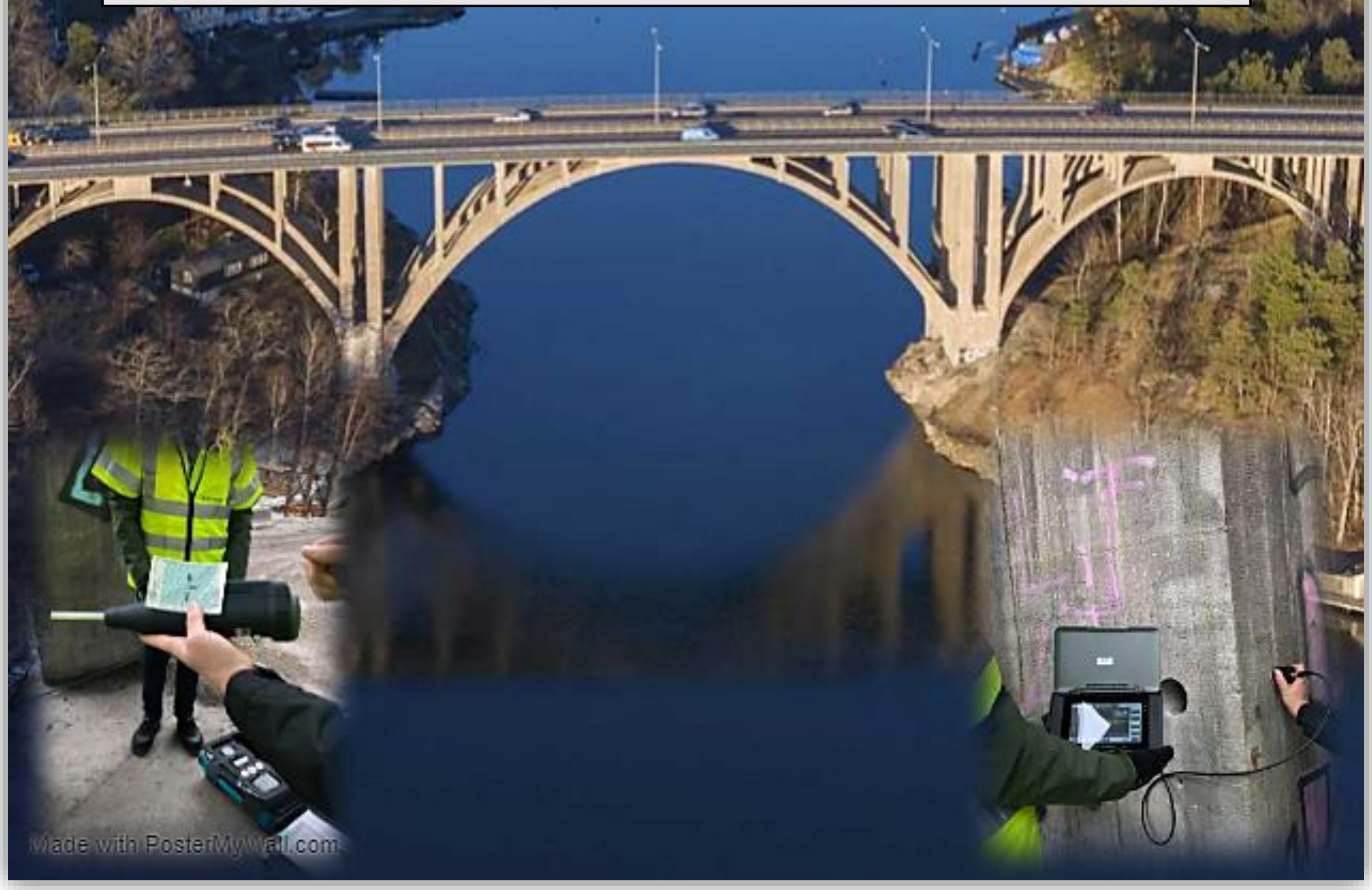

Abstract

It is inevitable that Structures become older and their intended use changes or the structural codes regulations change. In some regions the damage from seismic activities is a possibility. It becomes, therefore crucial to assess the structural capacity of such structures. The purpose of this study is to assess the different methods used for testing and estimating the characteristic in-situ compressive strength which is the most vital parameter required in structural assessment.

The focus of the study is for existing structures where there is no prior knowledge about the concrete strength. This study first investigates and evaluates the merits and demerits of these methods for investigation of the condition of in-situ compressive strength of concrete in existing structures. A case study of the Skuru bridge that was built in 1914 was utilized for this study. The study is based on information of the construction data and some results from prior investigation performed by the company COWI. Afterwards, non-destructive tests were carried out with the UPV and Rebound hammer to assess the quality of the concrete.

In addition, the study assesses the use of different interpretation methods with regards to reliability and practical application. The results were interpreted in accordance to the European codes, Swedish codes and other interpretation methods. The difference of the results from the different interpretation methods are compared and evaluated for reliability and efficiency.

The test results confirmed that the concrete consisted of the same strength class. However, the results from the different interpretation methods are dissimilar. The reason for obtaining different results is because the methods depend on different methodologies. The study showed that some methods can sometimes overestimate the results and become unsafe for structural assessment. On the contrary, the other methods can yield lower but safer estimates.

Moreover, the use of small number of cores is evaluated for various methods. The reasons are because in practice, the preference is to avoid large number of cores. As a result, it is recommended to apply care and proper judgment in selection of the methods and interpretation of the results. It is also recommended to consider the methods with respect to the aim of the investigation, their limitations and assumptions.

Sammanfattning

Samtida befintliga konstruktioner blir äldre och de dimensionerade lasterna ökar med tiden. Ifatt med detta ändras även kraven för större laster. Därför bör regelbundna inspektioner och förbättringar genomföras. I vissa områden kan det även förekomma seismiska rörelser som i sin tur förorsakar skador på strukturer. Av bland annat dessa anledningar är det därför viktigt att bedöma bärförmågan för befintliga konstruktioner. Syftet med arbetet är att granska de metoder och tillvägagångssätt som finns för att kunna bedöma den karaktäristiska tryckhållfastheten för betong i befintliga konstruktioner. Tryckhållfastheten är den styrande parametern för materialet under tillståndsbedömningar.

Huvudfokuset med arbetet är att bedöma den karaktäristiska tryckhållfastheten för befintliga konstruktioner som saknar information om nuvarande tryckhållfasthet. Till en början utfördes en noggrann litteraturstudie för alla applicerbara metoder. Därefter gjordes en undersökning och bedömning av för- och nackdelar med vardera metod. Syftet med dessa metoder är att kunna mäta den nuvarande tryckhållfastheten för befintliga konstruktioner.

Efter en ingående litteraturstudie, valdes de icke-destruktiva metoderna Ultrasonic pulse velocity och Schmidt Hammer. Dessa metoder applicerades senare på Skuru bron i syfte att utföra icke-destruktiva tester för att bedöma betongens kvalitet. Tidigare konstruktionshandlingar samt provtagningsrapporter från Skurubron som byggdes år 1914, har legat till grund för detta arbete. Företaget COWI är ansvariga för Skurubron projektet och har tillhandahållit all information om bron.

Vidare, redovisar detta arbete olika beräkningssätt för samtliga metoder utifrån olika standarder och tolkningsmetoder. För varje tolkningsmetod har evalueringar och analyser utförts med avseende på tillförlitlighet och praktisk tillämpning. De redovisade beräkningssätten har använts för att räkna fram resultat från destruktiva och icke-destruktiva tester. Resultaten tolkades i enlighet med europeiska koder, svenska koder och andra tolkningsmetoder. Skillnaden mellan resultaten från samtliga tolkningsmetoder jämförs och utvärderas med hänsyn till tillförlitlighet och effektivitet.

Testresultaten från UPV och Schmidt Hammer bekräftade att betongen består av samma tryckhållfasthetsklass. Resultaten från de olika tolkningsmetoderna var dock olika. Anledningen till att det blev olika resultat beror på att varje tolkningsmetod utgörs av sina egna metodiska procedurer. Resultaten visade även att vissa standarder kan övervärdera resultaten vilket kan resultera i fel bedömning av den karaktäristiska tryckhållfastheten. Å andra sidan, resulterade vissa tolkningsmetoder i lägre men säkrare uppskattning av tryckhållfastheten.

Utöver detta, utfördes det beräkningar på de destruktiva testerna utifrån olika tolkningsmetoder. Beräkningarna baserades dels på att räkna på ett mindre antal kärnor. Skälet till detta är att man i praktiken vill undvika att borra ett stort antal kärnor. Resultaten visade att korrekt bedömning och försiktighet vid val av metod och tolkningsmetod behöver implementeras. Det rekommenderas även att överväga metoderna med hänsyn till utredningens ändamål, dess begränsningar och antaganden.

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Table of Contents

List of symbols and abbreviations

Abbreviations

R – Rebound hammer

Q – Q value

UPV – Ultrasonic pulse velocity

CSC – Concrete strength class

BO - Break off

GCSC - Graphical concrete strength class method

NDT- Non-destructive test

Symbols

D - spring constant [kN/m]

$E_{forward}$ *- the energy before the impact [J]*

$E_{reflected}$ *- the energy following the impact [J]*

X_0 *- displacement triggering the impact [mm]*

x_R *- displacement after the impact [mm]*

ρ *- Density [kg/m³]*

λ *- Wave length [mm]*

f – frequency [cycles/s]

P_{Bo} *– BO force at the top [KN]*

h- height of cylindrical core for break off test

M - Moment at the top of cylindrical core for break off test

D - diamater of cylindrical core of the break off test

S - depth of Neutral axis of the cylindrical core of break test

σ *- stress from the the pull-out load*

τ *– shear stress from the pull-out load*

D - bearing diameter of pull-out

d - insert diameter of pull-out

h - embedded depth of pull-out insert

f_p - pull-out strength [MPa]

F - pull-out force [N]

A - surface area [mm^2]

d_1 - diameter of the head of pull-out insert [mm]

d_2 - inner diameter of bearing ring [mm]

h - distance from the pull-out insert head to the concrete surface [mm]

f_c - the compressive strength [MPa]

F - the maximum load at failure [N]

A_c - the cross-sectional area of the core on which the compressive force acts [mm^2]

$f_{ck,is}$ – characteristic in-situ compressive strength [MPa]

$f_{m(n),is}$ – average in-situ compressive strength [MPa]

k_2 – 1,48 [-]

s – standard deviation of the samples [MPa]

$f_{is,lowest}$ – lowest value of core strength [MPa]

k – factor from Table 3 in EN13791:2007 [-]

$f_{c,m(n),is}$ - average in-situ compressive strength [MPa]

k_n – confidence number from EN 13791:2019 [-]

$f_{c,is,lowest}$ - lowest value of core strength [MPa]

M - value from Table 7 in EN 13791:2019 [MPa]

$f_{c,m(m)is}$ – average of strength values from correlation [MPa]

$f_{c,is,reg}$ – strength values corresponding to indirect tests from correlation [MPa]

$f_{c,is}$ – in-situ core strength [MPa]

m – number of indirect tests in a test region [-]

n – number of samples [-]

$t_{0.05}$ – t-score for 95% confidence limit [%]

s_n – standrad deviation of n samples [MPa]

R – spread limit [%]

f_{ckjust} – adjusted characteristic compressive strength [MPa]

f_{kk} – characteristic compressive strength [MPa]

$f_{ck,inf,is,cyl}$ - Lower characteristic in situ compressive strength of core results (5% fractile) [MPa]

$f_{ck,sup,is,cyl}$ - Upper characteristic in situ compressive strength of core results (95% fractile) [MPa]

f_{cm}- Average value concrete standard cylinder compressive strength at 28 days [MPa]

$f_{cm,is,cyl}$ - Average in situ cylinder compressive strength of core test results [MPa]

$f_{cm,is,cyl}(t_o)$- Average value of the in situ compressive strength of cylinder core results at the time t_o [MPa]

$\beta_{cc}(t^-)$ - Coefficient which depends on t^- [-]

t^- - Backward time propagation [s]

C_m - Most probable value for C [MPa]

C_{sup} - Upper bound for C [MPa]

C_{inf} - Lower bound for C [MPa]

C - Strength class for concrete at the age of the core drilling test [MPa]

C_d - Design value of C [MPa]

C_{70} - Set of CSCs for which is valid the probabilistic condition P(C [C70) = 70% [MPa]

C_0 - Strength class for concrete at the age of 28 days [MPa]

$C_{0,d}$ - Design value of C0 [MPa]

$C_{0,inf}$ - Lower bound for C0 [MPa]

$C_{0,m}$ - Most probable value for C0 [MPa]

$C_{0,sup}$ - Upper bound for C0 [MPa]

$C_{0,R}$ - C0 known from the original construction plans [MPa]

$C_{0,70}$ - Set of CSCs for which is valid the probabilistic condition P(C0 [C0,70) = 70% [MPa]

$f_{ck,cube}$ – In-situ cube characteristic compressive strength [MPa]

$f_{cm,cube}$ (Y) - Mean of the logarithm of in situ core test results [MPa]

n- number of cores [-]

s(Y) - Standard deviation of the logarithm of in situ core test results [MPa]

t_{n-1} - The value if t distribution for degree of freedom [-]

s_{min} - represents minimum standard deviation recommended from the results of experiments [MPa]

v_x – Coefficient of variation [-]

k_3 *– factor from DIN 13791:2017 [-]*

σ_{B28} *160 – The compressive strength of the columns and arch at the time of construction*

1 Introduction

The compressive strength in existing structures is the most important parameter required in appraisal of old structures (Alwash 2017). According to RILEM, the increase in structures showing signs of deterioration, has raised the interest in the area of testing the in-situ compressive strength to a great deal.

The design life of many bridges in the Europe is reached (Europa.eu 2019). For instance, 75% of the bridges owned by Trafikverket are 50 years old. Many of the bridges' design loads were lower than today's standard loads (Pantura project 2013). Further, the introduction of the high-speed railway requires the evaluation of the bridge's capacity for increased dynamic loads. Those factors are expected to increase the demand of assessing existing structures.

In this thesis the 'existing' structures refers to the condition where there is no prior knowledge about the compressive strength of the concrete.

The assessment of condition of existing structures can be performed for following reasons:

1) The evaluation of structural capacity of existing structures in order to plan for different use or for complying with new standards.
2) The assessment of seismic damages

At the beginning of an investigation, the assessment of the characteristic compressive strength is performed with various methods of testing. The selection of those methods depends on the purpose of the testing and the required accuracy of the results. Therefore, the proper planning of the test methods can enable smooth execution of the investigation. The different methods of testing have different advantages, disadvantages and limitations. The factors that affect the results have to be considered during and after the execution of tests.

The selection of appropriate methods of testing should be followed by interpretation methods that are practical and have reliable accuracy. The interpretation methods have different estimation procedures and requirements. Therefore, different interpretation methods produce different results.

Furthermore, the obtained interpretation results feature tradeoff between accuracy and economy. In professional practice the preference is to use small number of cores to minimize cost and avoid damage. Hence, the choice of the application of the methods needs careful assessment of the required safety level of the estimate and the implication of the costs.

The use of large number of cores is not practical for most of the cases. However, large number of cores are required to estimate the compressive strength using correlation. Additionally, the minimum number of cores required for estimation of characteristic compressive strength according to the new EN 13791:2019 has practical limitations because of the large number of cores required. On the other hand, the reliability of the use of small cores is questionable for all methods because of reliability issues. In spite of this fact, according to RILEM much of the current practice relies on taking small number of cores. In this study the estimation of the characteristic in-situ compressive strength is assessed for the application of different interpretation methods. Furthermore, the scatter of small number of cores for different cases of interpretation methods is evaluated. Sefrin and Weber studied the results of the different versions of EN 13791 and found out that the statistical methods used in some of the standards are not reliable.

In this study various testing and estimation methods are applied in a case study of Skuru bridge. The scatter of the estimation is presented and the methodological differences of the methods and their outcomes are analyzed. In addition, the effect of the small number of cores is evaluated for different methods by simulation studies combining the different cores to form different samples. We hope this limited thesis will help enhance the understanding about the discipline of assessment of existing structures significantly.

1.1 Research question and objectives

The aim of the thesis is to evaluate the different methods of testing and estimation of in-situ compressive strength of concrete in existing concrete structures. Various methods of testing are compared with respect to their advantages, disadvantages, and limitations. The scatter of the results of different methods of interpretation are analysed.

The research questions in the thesis are:

- *How to determine the characteristic in-situ compressive strength of concrete in existing structures?*

- *Which methods of testing are efficient with respect to practicality?*

- *Which interpretation methods are efficient with respect to practicality?*

- *How are the European codes, Swedish code, German code and Netherland study applied in practice?*

- *What are the differences between the test methods?*

- *What are the differences between the interpretation methods?*

1.2 Scope of thesis and limitations

This thesis includes literature study, methodology, results, discussion, recommendations and conclusion. The scope of the thesis is to assess various test methods and approaches of estimation of the characteristic in-situ compressive strength of existing structures.

The study is based on previous investigations performed on the Skuru bridge. The methods of testing were opted based on the availability and accessibility. The methods adopted to estimate the compressive strength are EN13791:2019, EN13791:2007, GCSC method, DIN EN 13791:2017 and a method based on a Netherland study.

The results obtained from each interpretation method are compared and presented. The thesis includes discussion on the methods, results and make valuable recommendations.

The following were limitations for this thesis study:

- Limitation of the instruments/equipment supply
- Cost
- Accessibility
- Few numbers of indirect tests

2 Literature study

2.1 Methods of testing of in-situ compressive strength

2.1.1 Non-destructive methods

2.1.1.1 Rebound hammer

The surface hardness of concrete is one of the factors that indicates the quality of concrete. In 1930s, masses activated with energy were caused to hit concrete in order to assess the concrete member's compressive strength. The surface dents were counted as measurement of the strength of the concrete. Later, the measurement of the rebound distance of the masses were adopted as measurement of the strength (Bungey et al. 2006).

The rebound hammer method is developed by the Swiss engineer Ernst Schmidt in 1948. The method was developed in the Swiss federal material testing and experimental institute of Zurich (Malhorta & Carino 2004).

Schmidt hammers are categorized into N type and L type according to the intensity of energy of the impact to the concrete. The N types are suitable for high energy impact and for members with sizes greater than 100 mm thick. The L types are suitable for low impact energy and for brittle concrete for member sizes less than 100 mm thick (Proceq 2020).

The other classification of Schmidt hammers concerns the way the surface hardness is measured. Accordingly, the hammers are classified into Original (R) or Silver (Q) types which are shown in *Figure 2.1*. The R-hammers measure the rebound distances and the Q hammers measure the velocities of the impact. Schmidt hammers that measure R-values are called original Schmidt hammers and those measuring Q-values are called silver Schmidt hammer.

The Q hammers have more accuracy and are applicable for wider compressive strength ranges (10-100 MPa) as shown in *Figure 2.2*. It is common that the Schmidt hammers in practice can be equipped with enhanced functionalities of electronic and software capabilities. Today, it is common to find Schmidt hammers which can store the data and perform post-processing of the data (Proceq 2020).

Figure 2.1: Original and silver Schmidt hammer (Proceq 2020)

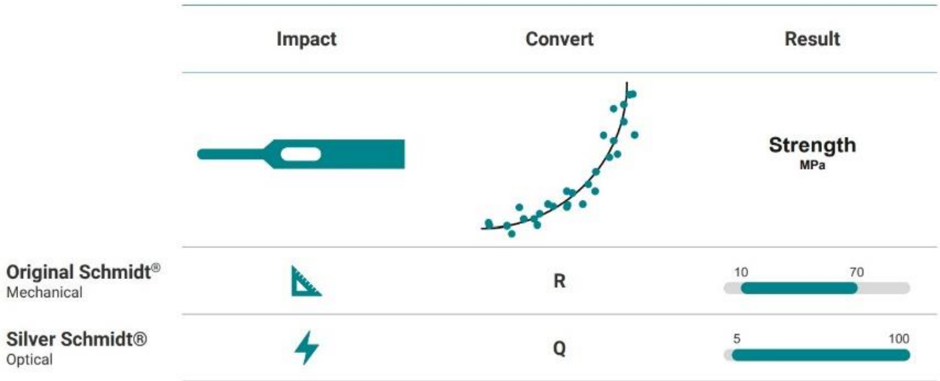


Figure 2.2: The ranges of strength applicable to Q and R type hammers (Proceq, 2020)

2.1.1.1.1 Principle and theory of method

The theoretical relationship between the rebound results and the compressive strength of concrete is difficult to justify. Hence, the accuracy relies on empirical correlations of the results with the actual strength of the crushed core samples (Bungey et al. 2006).

Schmidt hammers can weigh about 1.8 kg (*Figure 2.1*). The main components of a basic Schmidt hammer are a plunger, a hammer mass, a spring, a latching mechanism and a rebound scale. The latching mechanism locks the hammer mass to the plunger and the rebound scale registers the distance of the rebound of the hammer mass. The rebound scale ranges arbitrarily from 1-100. The measured value of the rebound scale is called the rebound number (Malhorta & Carino 2004). Modern Q-hammers have differential optical absolute velocity encoder, which measures the velocity accurately (RILEM 2012).

The rebound number is the output of the measurement of the Q and R types of Schmidt hammers. The theoretical computation of the rebound number is the percentage between the distance travel and the velocity of the mass before and after the impact. The following equation represents the theoretical relationship between the forward and rebound travel distances. Equation (2.1) depicts the calculation for the R-value hammer.

$$R = 100 * \sqrt{\frac{E_{\mathrm{reflected}}}{E_{\mathrm{forward}}}} = 100 * \sqrt{\frac{1/2Dx_{\mathrm{R}}^2}{1/2Dx_{\mathrm{o}}^2}} = 100 * \frac{x_{\mathrm{R}}}{x_o} \tag{2.1}$$

Where D is a spring constant, E_{forward} is the energy before the impact, $E_{\mathrm{reflected}}$ is the energy following the impact, x_{o} is the displacement triggering the impact and x_{R} is the displacement after the impact.

For the Q hammers the Rebound number is the percentage of the forward velocity of the rebound. Equation (2.2) represents the computation of rebound number for Q types.

$$Q = 100 * \sqrt{\frac{E_{\text{reflected}}}{E_{\text{forward}}}} = 100 * \sqrt{\frac{1/2mv_{\text{R}}^2}{1/2mv_{\text{o}}^2}} = 100 * \frac{v_{\text{R}}}{v_{\text{o}}} \tag{2.2}$$

Where the rebound value Q is expressed as a function of the respective kinetic energy before (E_{forward}) and after ($E_{\text{reflected}}$) the impact. The two quantities depend on the mass m of the hammer and on the respective velocities immediately before (v_o) and after (v_R) the impact (Breysse 2012).

The R value is the measurement of the energy of impact without the loss of energy during impact. However, in reality R-values are affected by friction on the guide rod, friction of the drag pointer on the slider scale, the influence of gravity, and the relative velocities between the unit and mechanical parts. Therefore, R values need to be compensated for the factors that are affecting the result. However, the Q value measurement is not affected by those factors and correction is not needed for friction or the impact direction. (Denys Breysse, 2012). The Q-hammers can also automatically account for the carbonation depth though the calibration is still required (RILEM 2012).

2.1.1.1.2 Method of testing

The test is performed by holding the hammer perpendicular to the surface of the concrete (A) (*Figure 2.3*). As the test starts, the body of the hammer is pushed towards the concrete (B). As the body is pushed towards the concrete surface, the mass of the hammer moves away from the concrete resulting in the stretching of the spring (C). When the limit of the upwards movement is reached the latch is automatically released. As the latch is released the energy stored in the spring propels the mass towards the concrete and produces impact with the surface. However, the hardness of the concrete surface makes the mass rebound and the sliding scale travels with the rebound and records the resulting distance (D) (Malhorta & Carino 2004).

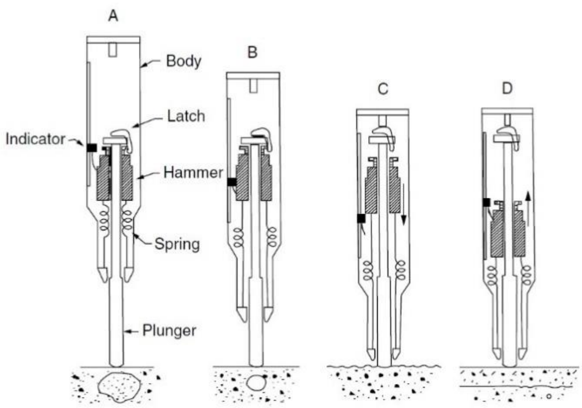


Figure 2.3: Schematic cut view of Schmidt hammer showing the operational stages A,B,C and D (Malhorta & Carino 2004).

2.1.1.1.3 Factors affecting test results

The readings show significant variation because of the differences in the local conditions of the concrete such as: presence of cracks, voids and the type of aggregate at the surface.

Therefore, a number of readings should be performed and the average of the readings is considered as the measurement (Bungey et al. 2006).

The results of hardness tests can also be affected by the characteristics of the mix, the member and the direction of application of the impact. (Bungey et al. 2006).

Although the effect of cement content on Portland cement is negligible, differences in other cement types may be significant. Super-sulfated cement and high-alumina cement can result in 50% and 100% stronger correlation strength than Portland cement. For coarse aggregates the correlation of strength depends on the type and source of aggregate. Cement paste gives higher rebound number and there is difference in results between different types of aggregates since their hardness varies. The results also differ for light weight and dense weight aggregates (Bungey et al. 2006).

The *Figure 2.4* Shows the influence of aggregate on the results.

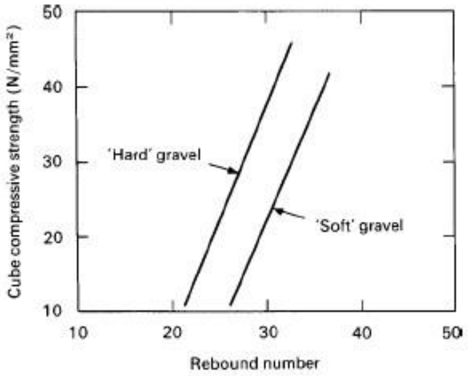


Figure 2.4: Influence of aggregate on the result (Bungey et al. 2006)

The effective mass of the member, slenderness, the boundary conditions and stress state affect results because of the vibration and movement that can be caused by the impact of the hammer. The test assumes full compaction since the performance of the test requires smooth and well-compacted surface. Troweled surfaces can result in overestimation of the results and the method is not suitable for open textured and exposed aggregate surfaces. The most favorable surfaces are those formed by formworks and other surfaces may require grinding to get representative results (Bungey et al. 2006).

Carbonation effects are insignificant for young concrete. However, the carbonation increases with the age of the concrete and can be as high as 20 mm for old concrete. Carbonation results in overestimation of results of the correlation due to formation of hard carbonated skin. The hardness of the surface of concrete is higher when dry. Wet surfaces can result in lower strength of about 20%. The effect of moisture in wet conditions should be accounted for during tests. Temperature effects are insignificant in normal practice of hammer tests. However, the effects of extreme temperatures on the readings need particular attention. The EN 12504-2 limits the temperature to 10-35°C (Bungey et al. 2006).

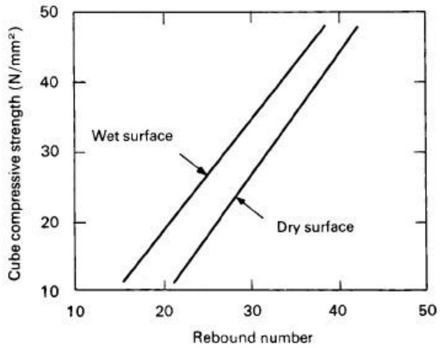


Figure 2.5: Influence of moisture content on the result (Bungey et al. 2006)

The direction of the impact on the concrete surface affects the test results. The most favorable condition is when the direction is horizontal. If the impact direction is up or down on horizontal surfaces or at inclined angles correction of the results for gravity are necessary (Breysse, 2012).

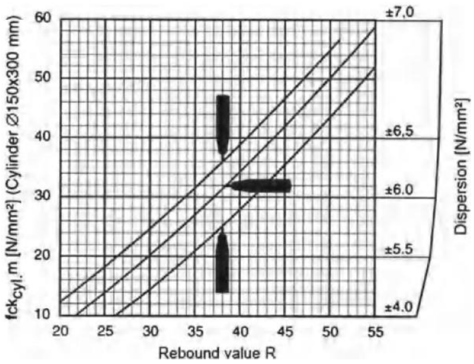


Figure 2.6: Influence of orientation of hammer on the result of original hammers

The presence of porous concrete hidden under the surface and very small cover of reinforcement can affect the result of rebound number (Brencich et al. 2020).

The use of hammer method should be accompanied by the consideration of the factors that account for the effects on the results. The use of hammer method should not be regarded as substitute of standard compression tests but rather a way of comparing different concrete samples and assessing the quality of concrete (Malhorta & Carino 2004).

2.1.1.1.4 Standards and correlations

The influences of the various variables mentioned in section 2.1.1.1.3 make it unlikely to adopt a general calibration curve. The calibration should be based on samples from the same type of concrete and under the same conditions of the concrete that is to be investigated. The number of factors that could affect the result makes the method least reliable for determination of compressive strength. The accuracy of the results obtained depend on the elimination of the influence of the various factors which are usually disregarded during calibration (Bungey et al. 2006).

The calibration of the compressive strength need correction for orientation of the impact for R type of hammers as shown in *Figure 2.6*. The carbonation can increase the measured strength as much as 50% more than the actual strength. The best way to account for the effects of carbonation is to correlate the results with the specimens from the specific concrete under test (RILEM 2012).

It is essential that the hammers are calibrated with standard anvil mass for the proper functioning. The calibration is necessary because the results can be changed due to wear of the mechanical parts (Bungey et al. 2006).

In existing structures, a properly calibrated device can give results with an accuracy of 30-40% of the in-place strength according to FHWA guide (FHWA 1997). According to Malhotra the results can have accuracy up to ± 25% (RILEM 2012).

There are several standards and guidelines that provide guidance on the procedures of the application of the methods and interpretation of the results.

-ASTM C 805, A standard test method for rebound number of hardened concretes ,1994

-EN 12504-2, Testing concrete in structures-Part 2, Non-destructive testing-determination of rebound number,2001

- EN 13791, Assessment of in-situ compressive strength in structures and precast concrete, Brussels,2007

-ACI 228-1R-03, In place methods to estimate concrete strength, Report aby ACI committee, 2003

The requirements for number of readings vary among the various standards. EN 12504-2 recommends a minimum of nine readings taken over an area not exceeding 300 mm square, with minimum spacing of 25mm from each other or from an edge. ASTM C805 (59) requires that minimum of ten readings per location (Bungey et al. 2006).

2.1.1.1.5 Advantages and limitations

The method is widely used because of its simplicity, speed and low cost (RILEM 2012). The rebound number is sensitive to change in quality of concrete and inadequate mixing or segregation. That makes it a reliable and quick method for checking of uniformity of concrete. It enables low-cost assessment of quality with smaller number of drilled cores. The tests results are more consistently reproducible than any other NDT method. The method is also suitable for determining the areas of poor concrete quality for optimizing the number of cores (Bungey et al. 2006). The method has a stronger connection with the mechanical properties than any other NDT technique (RILEM 2012).

The main disadvantage of the method is the poor reliability in estimation of compressive strength due to influence from various factors affecting the results as mentioned earlier. Moreover, the method only enables the assessment of the mechanical property near the concrete surface. The estimation of strength requires calibration with destructive tests. Even with calibration it is not reliable to depend only on this method (Bungey et al. 2006).

The method cannot be used on frozen concrete surfaces. Slenderness of the member can limit the applicability of the method. EN 12504-2 recommends that a member should be at least 100 mm and firmly fixed in the structure.

2.1.1.2 Ultrasonic pulse velocity

The interest of determining the properties of concrete without destructing the material, has been a universal desire (Malhorta & Carino 2004). The first two countries that developed the pulse velocity after World War II were England and Canada (Alwash 2017). During 1960s, the pulse velocity methods moved from being tested in laboratories to become used on site. Ever since this method was developed, various countries have begun to introduce and standardize this procedure (Malhorta & Carino 2004).

The ultrasonic pulse velocity (UPV) method is a non-destructive and acoustic method that is used for several purposes such as evaluation of concrete quality regarding its compaction, Young-Modulus and texture. Further on, this method is also used for evaluating the cracks, the compressive strength, and the characterization of the concrete (Helmerich et al. 2007).

This technique is a stress wave propagation method that generate wave pulse velocities with an electro-acoustic transducer through the concrete (Alwash 2017). The transducers are placed on each side of the concrete to send ultrasonic waves from one side of the concrete to the other. The wave velocity is then calculated by measuring the duration of flow time for the waves to pass through the concrete (Karahan et al. 2020).

The UPV is a fast, easy and popular non-destructive method. The method is also considered to be successful for verifying the quality and strength of concrete in distinct parts of the structure component or in the structure itself (Ariöz et al. 2009).

2.1.1.2.1 Principal and theory

There are different types of mechanical wave propagations during the application of an impulse to a medium and large surface. These wave types consist of longitudinal waves also called compressional or P- waves, transverse waves, also called shear or S-waves and Rayleigh waves, also called surface or R-waves. These various waves have dissimilar velocities, the quickest waves are the compressional waves and the lowest waves are the surface waves (Alwash 2017; Malhorta & Carino 2004).

The P-waves spread through the robust medium in a way that is analogous to sound waves that distribute in the air. The velocities in concrete that belong to the shear and surface waves are approximately about 55 to 60 % respectively of the P-waves velocities. The density and elastic properties of the material determine the particular velocity of the wave. For a solid, isotropic, homogenous and elastic medium, the compressional wave velocity is calculated according to equation (2.3):

$$V = \sqrt{\frac{KE}{\rho}} \quad \text{(km/s)} \qquad \text{where:} \qquad (2.3)$$

V= Compressional wave velocity

$K = \frac{(1-v)}{(1+v)(1-2v)}$

v = Poisson’s ratio

E = Dynamic modulus of elasticity [N/mm^2]

p = Density [Kg/m^3]

(Bungey et al. 2006; Malhorta & Carino 2004).

In the above formula, the K-value is comparatively insensitive to deviations of the Poisson's ratio, hence the variation in p (density) and E (Elastic modulus) give more substantial effect on the compressional wave velocity (V) (Bungey et al. 2006; Malhorta & Carino 2004).

Additionally, the arrival time (t) of the compressional waves from the transmitting transducer to the receiving transducer (according to *Figure 2.7*) could be measured. Similarly, the distance between the transducers, path length (l), is measured and finally the UPV of longitudinal waves, Vp, is easily calculated according to equation (2.4):

$$Vp = \frac{L}{t} \tag{2.4}$$

(Abbas Alwash 2017).

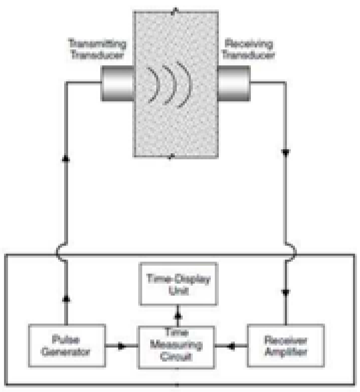


Figure 2.7: Schematic diagram of UPV (Alwash 2017).

Finally, the velocity of the propagating wave motion is related to the wavelength λ and the frequency f: V = λf. The wavelength l, is ascribed as a distance e.g., mm and the frequency f, is ascribed as hertz or cycles/s. An increase in the wave frequency results in a decrease for the wavelength and contrariwise. For the concrete material the higher limit of compatible frequency is approximately 500 kHz with a measured wavelength of 10 mm. These numbers are in the range of the coarse aggregate particles for concrete. A frequency of 20 kHz can traverse 10 m, meaning that larger pathlengths can be crossed with lower frequencies (Malhorta & Carino 2004).

To run the UPV test, the equipment comes with one transducer, one receptor, pulse generator, an electronic timing device and an amplifier. The soundwaves go from the transmitter to the receptor and the electronic timing device measure the time interval passing between the transducers (Branco & Brito 2004; EN 12504-4:2004). *Figure 2.8* presents the development of transducers that has been taking place during decades

For the test running on concrete, transducers with frequency ranges from 25 to 100 kHz are being used. Different resonant frequencies are applied to different sizes and different properties of the concrete specimens. For small sized specimen, high-strength concrete and short path lengths, high frequency transducers (higher than 100 Hz) are required. For large specimens, concrete with large size aggregates and longer paths, low frequencies are used (lower than 25 Hz) (Malhorta & Carino 2004).

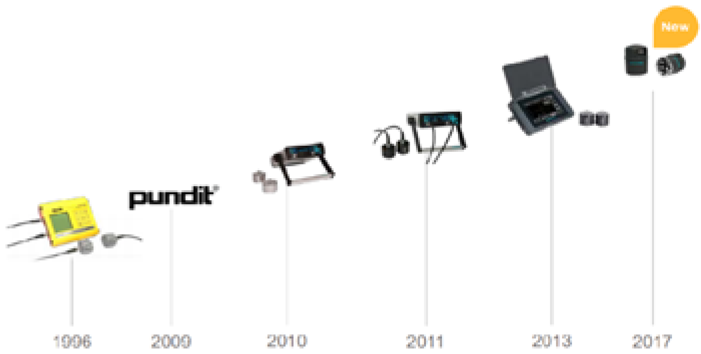


Figure 2.8: Development of transducers

(Proceq)

2.1.1.2.2 Method of testing

The basic concept behind the method is to measure the time required for the sound waves to go from one side of the concrete to the other (Branco & Brito 2004). During the time the sound waves travel through the concrete and reach the receptor, it transforms into a convoluted waveform that includes reflected shear waves or compressional waves. The compressional waves are the fastest, therefore they arrive first at the receptor (Malhorta & Carino 2004).

During the testing, it is essential to ensure complete connection between the concrete surface and the transducers. A thin layer of connection medium is applied to the concrete surface to make sure perfect coupling between the concrete and transducers. Afterwards, the transducer and receptor are pressed against the surface of the concrete to finally record the transit time of the soundwaves. The transit time reading is repeatedly recorded to avoid errors. Finally, the distance between the transducers is measured in order to calculate the ultrasonic pulse velocity (Alwash 2017).

The arrangement of the transducers can be accomplished in three different configurations (EN 12504-4:2004):

- The transducers are placed on opposite sides of the concrete surface, called direct transmission, see *Figure 2.9.*

- The transducers are placed between adjacent surfaces, called semi-direct transmission, see *Figure 2.10.*

- The transducers are placed on a single surface, called indirect or surface transmission, see *Figure 2.11* (RILEM 2012).

The difficulty with the first configuration is the access to the opposite surfaces of the concrete, but still, this method is the most accurate one and should be chosen. The second configuration, semi-direct transmission, is effortless to use but the challenge here is to define the distance between the transducers. The direct transmission technique is a more accurate choice than the semi-direct. At last, the indirect technique is easily applied for in-situ configuration since the accessibility to one face is facile, but this technique is less accurate (RILEM 2012).

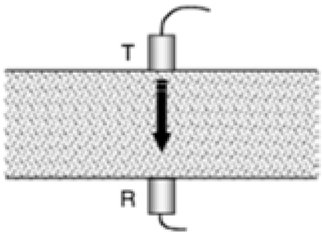


Figure 2.9: Direct transmission (Malhorta & Carino 2004).

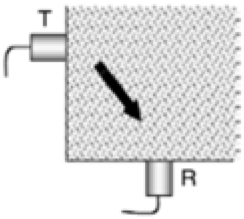


Figure 2.10: Semi direct transmission (Malhorta & Carino 2004).

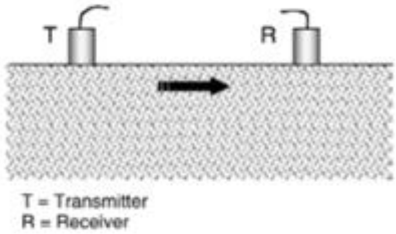


Figure 2.11: Indirect (Malhorta & Carino 2004).

2.1.1.2.3 Factors affecting test results

There are factors that affect the UPV test result and therefore it is important to make sure that the pulse velocity readings are reproducible (Malhorta & Carino 2004). Further on, the factors that affect the testing are the aggregate, water cement ratio, cement type, the coupling between the concrete and transducers and the presence of reinforcement.

The aggregate content affects the relationship between the compressive strength of concrete and pulse velocity, meaning that a high aggregate substance results in high pulse velocity. The water content results in lower compressive strength of concrete and at the same time increases the pulse velocity. Additionally, the cement type and the age of the concrete could impact the compressive strength and the pulse velocity too. All these factors generate ambiguity in the interpretation of the UPV results (Trtnik et al. 2008).

As mentioned earlier, there should be enough coupling between the concrete and transducer. That is necessary to prevent formation of air pockets that may occur and cause errors in the results. Furthermore, that is something vital to deliberate since only a minor amount of energy is enough to disturb the results. To eliminate these air pockets and guarantee proper contact between the surface and the transducers, a thin layer of petroleum jelly could be used, which has also been confirmed to be a great coupling (Bungey et al. 2006).

In order to reduce the measurement uncertainty, the test is replicated at different positions in a small area where the test is performed (Alwash 2017). The transit time should be repeated until a minimum value is achieved (EN 12504-4:2004).

This technique necessitates accurate measurement with the instrument being used, because the transit time that is measured is very short. The UPV technique is based on the measurement of sound waves, therefore any interfering wave can disturb the measurement. Therefore, adequate care is needed to realize accurate readings (Malhorta & Carino 2004).

In addition, the location of the reinforcement affects the duration of the waves. In places where the reinforcement is placed, the duration upsurges noticeably. Further, there are difficulties that arise when this method is being used on an existing bridge structure due to the location of the load-bearing elements. Semi direct transmission or indirect transmission are performed when it is impossible to conduct direct transmissions. This will thus increase the wave flow and consequently decrease the velocity of the wave propagation (Karahan et al. 2020). It is therefore better to perform the test readings in areas where the reinforcement is not located, or else correction factors must be applied (Malhorta & Carino 2004).

2.1.1.2.3 Standards & Correlations

The European and American standards do not specifically mention how many readings should be performed, rather it declares that enough readings should be completed in order to obtain the minimum value of the transit time. The American standard (ASTM C597, 2002) indicates that the indirect transmission should be avoided due to the uncertainty of the measured path lengths and that the calculated velocity is affected by the surface layer of the concrete.

The properties of concrete that influence the pulse velocity are the density and elastic modulus. These properties are in turn related to water cement ratio, age of concrete and type of aggregate. The strength of concrete is most affected by the water cement ratio and it affects the strength more than the type of aggregate. Thus, correlation of the strength of the concrete to the pulse velocity should be performed for a specific concrete mix. So, to assess the strength of the concrete using the pulse velocity for unknown concrete is not reliable (EN 12504-4:2004). Correlations with regard to different w/c ratios are illustrated in *Figure 2.13* and correlations to different aggregate types are illustrated in *Figure 2.14.*

The correlation is done by performing the UPV tests at the same location of the drilled cores in order to obtain a data set of pairs of results (EN 12504-4:2004). An example of correlation between velocity and cylinder compressive strength is illustrated in *Figure 2.12.*

It is worth mentioning that some concrete bridges have w/c ratios beyond the ranges illustrated in *Figure 2.13* and this would result in a more complex situation. It would then be better to apply another method for assessing the strength of the concrete (Sangiorgio 2021).

From a statistical point of view, the 95% confidence limit for the compressive strength is around ± 20 % of the mean value performed on one test (Bungey et al. 2006). Moreover, according to Bungey et al. 2006, the reliability of the absolute strength correlation is poor and this test method should be combined with other methods for evaluating the compressive strength of the concrete.

At last, one must consider the path lengths and aggregate sizes to perform the UPV test. There are two limitations that are deemed necessary:

Path length of 100 mm for concrete having maximum aggregate size of 30 mm

150 mm for concrete having maximum aggregate size of 45 m (Bungey et al. 2006).

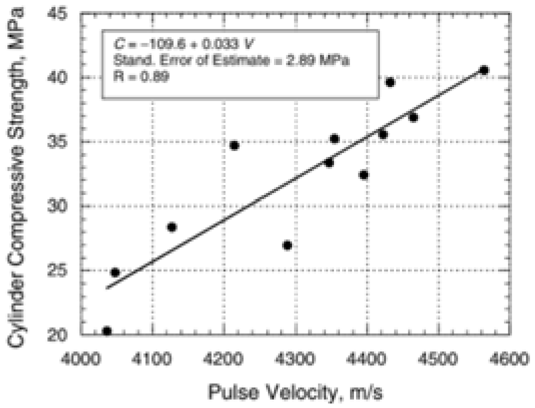


Figure 2.12: Correlation of pulse velocity and compressive strength (Malhorta & Carino 2004).

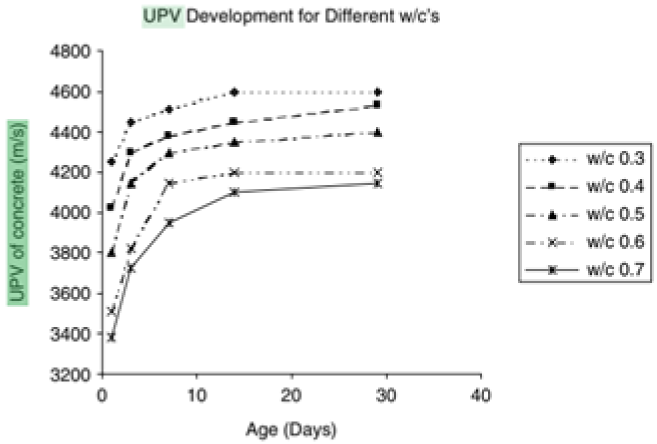


Figure 2.13: Correlation to different w/c ratios (Malhorta & Carino 2004).

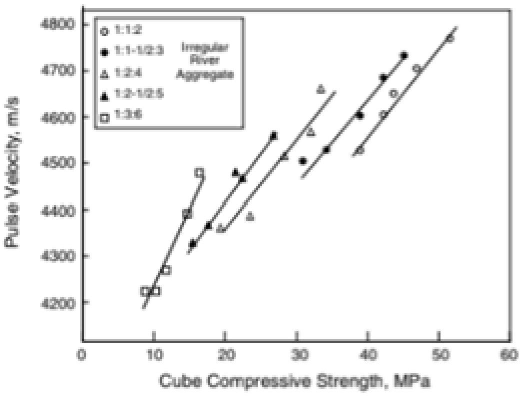


Figure 2.14: Correlation to different cement type- fine and coarse aggregate type (Malhorta & Carino 2004).

2.1.1.2.3 Advantages & limitations

The main advantages with UPV are the ease of application, non-destructiveness and low operational costs. The limitations of the method are the reliability of the correlation, accessibility to two opposite surfaces and temperature variations that must be considered. The temperature variations between 5 and 30°C have insignificant effect on the readings, but temperatures outside this range need to be considered in calculations using correction factors. Additionally, the absolute strength of correlation to the drilled cores is poor (Bungey et al. 2006; Alwash 2017)

2.1.2 Semi destructive methods

2.1.2.1 Penetration resistance method

Penetration resistance test is a type of surface hardness test; however, more depth of the concrete is assessed than the rebound hammer method. In the professional practice, the tests are known by the manufacturer name called Windsor. The methods cause localized failure which does not affect the structural strength of the member. There are two types of tests namely, probe penetration test and pin penetration test. The correlation to compressive strength is not affected by many factors as rebound hammer and ultrasonic pulse velocity tests. The advantages of this method are that the results can be immediately available and the test is less damaging and disruptive (Bungey et al. 2006).

Steel probes are utilized in the probe penetration test while steel pins are used in the pin penetration test. The steel probes are driven with high energy actuated by powder and the probe penetration depth is affected by the concrete strength and type of aggregate. However, steel pins are driven by low energy actuated by spring and the intention of the test is to only assess the mortar. If the aggregate is encountered, the result should be disregarded (Malhorta & Carino 2004).

Like other surface hardness methods, penetration test results indicate the relative strength of concrete in a structure. The determination of absolute strengths is possible only by correlation with actual strength of the same concrete determined by destructive tests (Malhorta & Carino 2004).

2.1.2.1.1 Windsor probe test

The use of penetration resistance tests became well established after the development of a device called the Windsor probe. The device was developed during 1964-1966 jointly between Port of New York authority and the Windsor machineries co. in USA. The aim of the development was to measure the compressive strength by the depth of penetration of probes by powder actuated drivers (Malhorta & Carino 2004).

The Windsor probe test measures the surface hardness of the concrete and the result relates not only to the compressive strength in the localized area but also the sub surface strength of the concrete. The method is used for the estimation of concrete strength and quality by measuring the depth of penetration of the probe driven in to concrete by a powder actuated driver (Malhorta & Carino 2004).

Figure 2.15: Windsor probe test kit (James instruments 2020)

2.1.2.1.1.1 Principle and theory of the method

The penetration of the probe subjects the concrete to complex dynamic compressive, tensile and shear stresses. This makes the theoretical explanation of the relation of mechanical properties with the stresses very difficult. As per the suggestion by Windsor equipment manufacturers, a subsurface compression bulb is responsible for resistance of penetration. The surface becomes crushed by the tip of the probe and the shockwaves cause spalling during penetration. Hence the kinetic energy is absorbed by crushing at the tip, by the friction along the probe and the compression of the concrete in the bulb.(Bungey et al. 2006).

However, it is suggested that the energy that is dissipated by the compression of the concrete in the bulb has a higher percentage of the energy before impact compared to that which is lost due to friction and crushing. *Figure 2.16* illustrates the compression bulb formed during the test. Although it is not proven, the claim is consistent with the reasonable assumption that the result of the measurement relates to the property of the sub-surface concrete rather than the surface concrete (Bungey et al. 2006).

The range of strength that can be measured by the probe penetration test is reported to be 40-80 MPa (Bungey et al. 2006). It was also found from experimental tests that the probes can break or bend if the strength is more than 80 MPa (Pascal 2000). However, the manufacturers claim the range of use as 10-110 MPa for the silver type of probes, which are utilized for high strength concrete (James instruments manual).

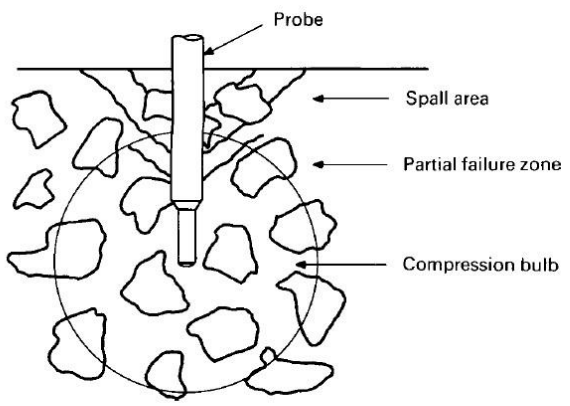


Figure 2.16: Compression bulb (Bungey et al. 2006)

2.1.2.1.1.2 Method of testing

The parts that constitute Windsor probe are a powder-actuated gun or driver, hardened alloy-steel probes, loaded cartridges and a gauge which measures the depth of penetration of probes. *Figure 2.15* shows the test kit of a probe penetration test. The probe tips (*Figure 2.17*) have different diameters for use with light concrete or normal concrete. The probe is driven by the firing of the powder charge which propels the probe with an energy of 79.5 m-kg. The amount of powder is adjusted as per the strength of the concrete (Malhorta & Carino 2004).

The procedure of the application of Windsor probe is simple. The surface that receives the probe must be smooth. If the surface is coarse, it should be grinded to smooth texture. The actuator is prepared according to the manufacturers guidelines and the test is performed according to relevant standards. The probe is driven through the holes of the locator plates that are placed on the concrete surface. After the probe's penetration is complete, the area is cleaned of debris and another plate is placed on the surface of concrete as shown in *Figure 2.18*. The measurement of the exposed length of the probe, commences by placing the calibrated measurement guage beside the probe. The measuring guage can be manual or electronic depending on the type of the equipment (Malhorta & Carino 2004).

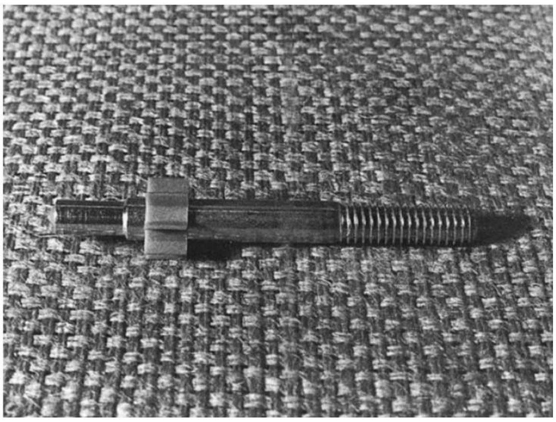


Figure 2.17: Penetration resistant probe (Bungey et.al. 2006)

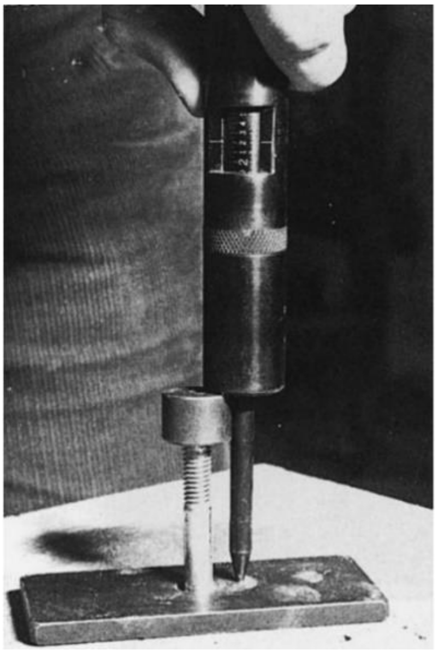


Figure 2.18: Measuring height (Bungey et.al. 2006)

There are two power settings which are related to two types of probes. The low power setting is used with gold type of probe that is applicable to light weight concrete with strength up to 19 MPa. Similarly, the silver probes can test high strength concrete up to 110 MPa (James instrument Windsor probe manual).

2.1.2.1.1.3 Factors affecting test results

The hardness of aggregate is an important factor that affects the results of the test. Therefore, the hardness of the aggregate is used as an important factor for calibration of the results which the manufacturers commonly include in the equipment manuals. However, the type of aggregate also affects the results significantly. It is observed that crushed aggregates result in higher strength than rounded aggregates. The difference in strength of bond at aggregatematrix surfaces can affect the depth of penetration. Moisture content, aggregate size (up to 50mm) and aggregate proportion have smaller effect than aggregate type and hardness. *Figure 2.19* illustrates the effect of aggregate type on probe results. The results should be calibrated with the same aggregate type of the concrete under the test (Bungey et al. 2006).

Carbonation can change the mechanical characteristics of the concrete to a certain depth and consequently the probe results may get affected. It is observed that the strength results from probe tests become overestimated for old concrete. That may be due to the microcracking between the cement paste and the aggregate. Moreover, the stress history of concrete can result in overestimation of the strength because of the cracking from the service loading. In both cases, the higher strength result occurs as a result of the effect from the above-mentioned phenomena on the compressive strength tests. Yet, the effects result in negligible probe results (Malhorta & Carino 2004).

Surface conditions such as texture and moisture content do not affect the results. However, hard surfaces can give non-representative low penetration values (ACI commite 228 report).

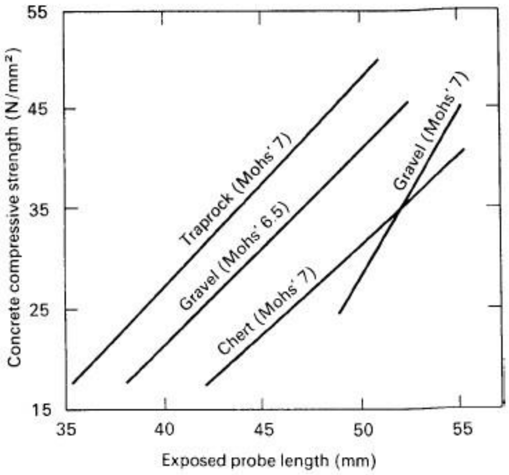


Figure 2.19: Influence of aggregate type (Bungey et.al. 2006)

2.1.2.1.1.4 Standards and correlations

The development of the correlation between the penetration and strength is difficult because of the two power levels. This makes it necessary to prepare separate calibration curves for the two power levels (Bungey et al. 2006).

Manufacturers provide correlation curves for estimating the compressive strength, however those curves are not reliable. It is necessary to correlate the results of the test with the type of concrete being investigated (Malhorta & Carino 2004).

A calibration of strength from the manufacturer can be presented in table forms for low and high-power tests as illustrated in *Table 2.1* or as graphs shown in *Figure 2.20*. The tables further indicate the strength for different Mohr's hardness of aggregates (James instruments 2020).

The standards that cover the test include:

-ASTM C803/C803M- The standard initially was issued in 1982 and later the part regarding pin penetration test was added in 1990

-BS 1881-207- The BS standard states that 3 readings are required at a location. The 95% confidence limits are ±20% .

Table 2.1: The strength for different Mohr's hardness of aggregates.

Metric Standard Power Strength Table (Cylinder Correlations)					
Compressive Strength (MPA)					
Exposed Probe (mm)	Mpa Mohs' N0. 3	Mpa Mohs' N0.4	Mpa Mohs' N0.5	Mpa Mohs' N0.6	Mpa Mohs' N0.7
35.0	19.7	14.8	-	-	-
35.5	20.6	15.8	10.8	-	-
36.0	21.5	16.7	11.8	-	-
36.5	22.4	17.7	12.8	-	-
37.0	23.3	18.6	13.8	7.3	-
37.5	24.2	19.6	14.8	8.4	-
38.0	25.1	20.5	15.8	9.4	-
38.5	26.0	21.5	16.8	10.5	3.5
39.0	26.9	22.4	17.7	11.6	4.7
39.5	27.8	23.4	18.7	12.7	5.9
40.0	28.7	24.3	19.7	13.8	7.1
40.5	29.6	25.3	20.7	14.9	8.3
41.0	30.5	26.2	21.7	16.0	9.5
41.5	31.5	27.2	22.7	17.0	10.7
42.0	32.4	28.1	23.7	18.1	11.8

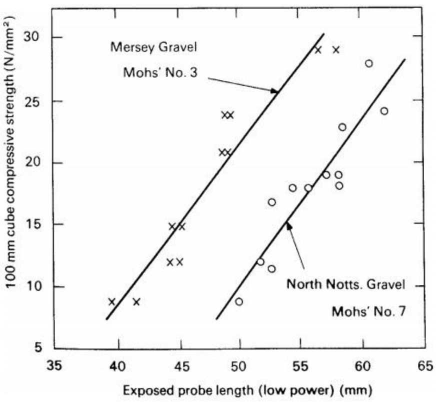


Figure 2.20: Typical calibration for low power range (Bungey et.al 2006)

2.1.2.1.1.5 Advantages and limitations

The test has high combined rating with respect to reliability, simplicity and economy and its correlation is affected by small number of variables (Malhorta & Carino 2004). Further, the test is not affected by operator skills. The variability indicated is a COV of about 5% and a coefficient of correlation of 0.98 for a single set of three tests (Bungey et al. 2006).

However, the test causes minor surface damage which may require patching. Although the method has many inbuilt safety measures, the operation of the equipment requires wearing of safety protection (Malhorta & Carino 2004).

The method gives more direct assessment of concrete at larger depth than rebound hammer. In addition, the results are comparable to accuracy of small dimeter cores and could be used as an option to coring (Buney et al. 2006).

The application of the method has limitations of minimum edge distance to avoid cracking of concrete. The test should satisfy the requirements of minimum distance between the probe locations and the minimum size of the structural members. The distance of the reinforcement especially less than 100 mm could have effect on the depth of penetration (Malhorta &

Carino 2004).

As mentioned earlier, the method can indicate the strength at larger depths of concrete, however the limitations of edge distances means that the method cannot be used as replacement of rebound method unless the results from rebound are unsatisfactory (Bungey et al. 2006).

2.1.2.1.2 Pin penetration method

The method was studied by Nasser and Al-Manaseer in the 1980s for determination of removal of formwork. This method is standardized in ASTM C803 in 1990 as an addition to the standard released in 1982.

The kinetic energy that drives the pin probe is 1.3% of the energy that powers Windsor probe. The low energy released cannot break aggregates and therefore the test can only measure strength of the mortar in the concrete. The test also is not sensitive for compressive strength above 28 MPa which limits its use for such ranges of strength (ACI committee 228 report).

2.1.2.1.2.1 Method of testing

The apparatus of the testing equipment consists of a pin within the shaft of the body of the tester as illustrated in *Figure 2.22*. The pin is held against a spring that is compressed during test preparation. As the spring is released on commencement of the test, the pin is driven in to concrete. *Figure 2.21* depicts a pin penetration test. The penetration of the pin into the concrete is related to the strength that the concrete has attained (Malhorta & Carino 2004).

Figure 2.21: Pin penetration apparatus (James instruments).

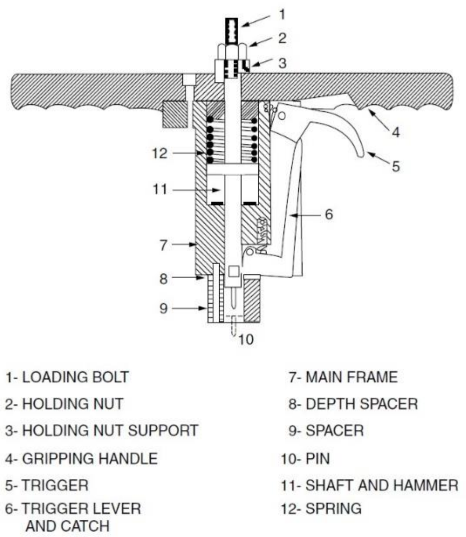


Figure 2.22: Schematic diagram of pin penetration testing apparatus (Malhorta & Carino 2004)

2.1.2.1.2.2 Standards and correlations

The method gives good accuracy of correlation for lightweight concrete between 3.1-24 MPa. The test is considered as the only test that does not require correlation for lightweight concrete (Bungey et al. 2006). *Figure 2.23* presents an example of correlation between pin penetration depth and compressive strength.

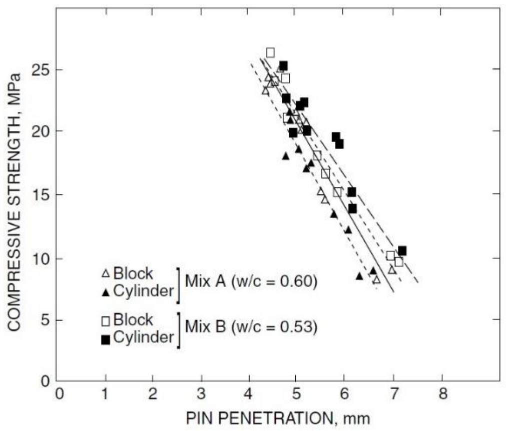


Figure 2.23: Correlation of pin penetration with compressive strength (Malhorta & Carino 2004).

2.1.2.1.2.3 Advantages and limitations

The method has advantages of speed, simplicity, low cost and low damage. The depth of penetration is unlikely to be larger than 8 mm, hence it is not affected by reinforcement depth. The limitations of the method are the applicable strength range, the type of aggregate and mix type. There is no known effect of temperature. However, carbonation poses difficulty in strength determination as shown by variability of COV of 18% in carbonated or old concretes (Bungey et al. 2006).

2.1.2.2 Break off method (BO)

The break off test was developed in Norway in 1976 by Johansen. Johansen's research indicated the method as a way of testing the in-place strength for form removal. In 1977 researchers at NTH and the Research group for cement and concrete in Norway developed and patented the method. (Malhorta & Carino 2004)

In 1984, Darl-Jorgegsen and Johansen reported that the results from BO test can detect variability in curing better than the pullout method. In 1979, Johannsen published a paper on the use of BO method on asphalt pavements made of vacuum concrete. The report concludes that the results of BO tests are comparable to that of flexure beam test.

In the same year, Carlsson studied the BO methods use in the field and concluded that the method had gained acceptance in the field work. In addition, Naik recommended the drilled core method as the preferable method (Malhorta & Carino 2004).

The *Figure 2.24* shows a test kit of Break off test.

Figure 2.24: Break off testing apparatus (James instruments)

2.1.2.2.1 Method of testing

The BO tester set comprises of the load cell, a manometer and a manual hydraulic pump. The load cell can be adjusted for high and low strength concrete. There are two different types of tests depending on the preparation of the cylindrical core. In the first type of the test a sleeve is inserted into fresh concrete to form the cylindrical core. Whereas for the other type of test, a cylindrical core is drilled. The testing apparatus is illustrated in *Figure 2.25*. The sleeve should be inserted by twisting and rocking action at spacing of 150mm. After the necessary depth is reached, the BO specimen should be tapped at the sides and at the top to ensure that the compaction and the localized conditions are not affected. In the case of the sleeves, the concrete bleeding is a problem (Malhorta & Carino 2004).

During the test the sleeve is removed except the sleeve ring and the debris is cleaned from the cylindrical slit and groove. The load cell is placed in the groove and the load is applied slowly until the specimen is broken off. The manometer reading can be correlated with the compressive strength of the cores from the concrete under investigation (Malhorta & Carino 2004).

In the case of drilled core tests, the concrete surface should be smooth enough to fix the vacuum plate of the drilling machine. The drilling core barrel should be maintained vertically during drilling until a depth of 70 mm is reached. A groove is provided at the top for placing the load cell (Malhorta & Carino 2004).

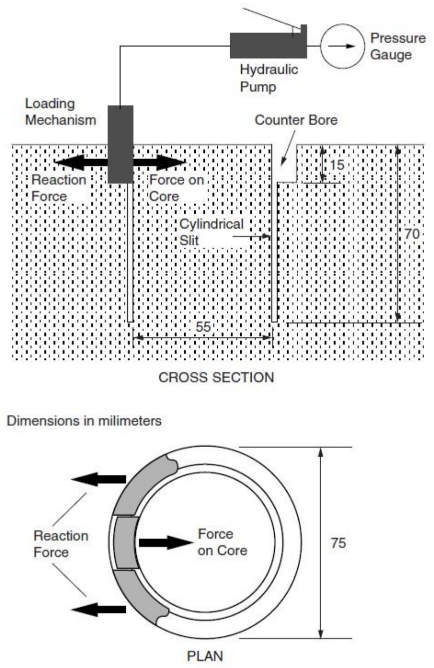


Figure 2.25: BO test apparatus

2.1.2.2.2 Principle and theory

The break off method is dependent on breaking of the cylinder core parallel to the surface. The cylindrical core is subjected to a force at the top of the cylinder which creates a cantilever moment at the bottom since it is free at top. The moment creates a stress that increases from the top to the bottom. The maximum stress is situated at the extreme fiber at the base of the cylinder. The stress at the extreme fiber of the bottom of the cylinder core is computed theoretically according to equation (2.5):

$$f_{BO} = {}^{M}/_{S} \qquad \text{where:} \qquad (2.5)$$

where:

$M = P_{BO} * h$

P_{BO} = BO force at the top

$h = 65.3$ mm

$S = {}^{(d)}/_{32}$

$d = 55$mm

BO method is the only method that directly measures the flexural tensile strength. Pull out method is another similar method that measures the tensile strength directly (Malhorta & Carino 2004).

2.1.2.2.3 Factors affecting test results

Barker and Ramirez reported a variation of 6,1% and 7,6% for sleeve and drilled core tests with respect to changes in water cement ratio, aggregate shape and size. Moreover, Carlsson and Naik have reported that crushed aggregate gives 10% higher strength than coarse aggregate (Bungey et al. 2006). On the contrary, BO reading is not affected by temperature, surface conditions and shrinkage (Malhorta & Carino 2004).

In 1987 Naik studied the effects of the tests made with sleeve and drilled core for high strength concrete. Accordingly, the results from the drilled core test were 9% higher than those from the sleeve test (Malhorta & Carino 2004).

2.1.2.2.4 Standards and correlation

A study showed that both methods of tests depicted good correlation with compressive strength and the results of the BO readings showed uniformity. Even though both methods showed acceptable correlation, Naik recommended the drilled core method as the preferable one (Malhorta & Carino 2004).

Furthermore, the calibration of BO readings with respect to core compressive strength should be prepared for the specific type of concrete. The *Figure 2.26* depicts an example of correlation curve of BO test. The manufacturers calibration curve does not reflect adequately the inherent properties of the concrete (Malhorta & Carino 2004).

The method was standardized in Sweden, Norway and Britain in 1982. The ASTM standard was withdrawn in 1992. Currently, the BS 1881 part 207 applies to standard procedures of application of the test (Malhorta & Carino 2004).

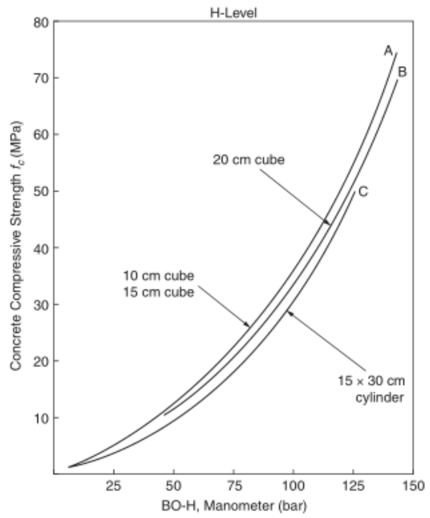


Figure 2.26: Correlation of BO manometer reading with compressive strength (Malhorta & Carino 2004).

2.1.2.2.5 Advantages and limitations

The peculiar feature of the BO test apart from other NDT tests is that it measures flexure strength directly. The method does not require pre-planning of test and requires only one exposed surface. The results have good correlation with compressive strength, acceptable accuracy and reproducibility of results (Malhorta & Carino 2004).

In addition, the method is quick, uncomplicated and the results are not affected by surface conditions, local shrinkage and temperature (Bungey et al. 2006).

However, the method may cause damage to the surface. The method is also limited by maximum aggregate size of 19 mm and a structural member with a minimum size of 100 mm (Malhorta & Carino 2004).

2.1.2.3 Internal fracture test

The test is developed by Chabowski and Bryden smith who were working with building research international in 1997. They were using the method to estimate residual strength of high alumina cement which became extended to Portland cement. Later the authors suggested an alternative method of loading, called pull force loading method to improve the accuracy of the results of torquemeter method of loading. The *Figure 2.27* illustrates a test apparatus of internal fracture test. These methods can offer a reliable means of testing when specially the concrete mix is unknown (Bungey et al. 2006).

Figure 2.27: Internal fracture test apparatus (James instruments)

2.1.2.3.1 Method of testing

At the start of the test, a hole is driven into the concrete with a depth of 35-40 mm and a width of about 6 mm. The hole is cleaned of dust and a wedge anchor bolt with 6 mm diameter and with expandable sleeve is inserted until it reaches 20 mm below the surface. First an initial load is applied to expand the sleeve, then the load is increasingly applied until the concrete fails by cracking. The peak of the graph that shows the relation of the load versus the bolt movement is taken as the failure load as shown in *Figure 2.29* (Bungey et al. 2006).

There are two methods for applying the load. In one of the methods the load is applied as torque by a torquemeter. In the case of the other method called pull force method, the load is applied as axial force by pulling. The result depends on the load application method. Further, the rate and the way the loads are applied affect the result (Bungey et al. 2006).

When the load is applied by torquemeter there is some twisting action that affects the results causing variability and decrease the failure load. The torquemeter is also insensitive during measurement and requires settling pauses in midst of test. Therefore, the authors developed the pull force method that is free of twisting action as shown in *Figure 2.30* (Bungey et al. 2006).

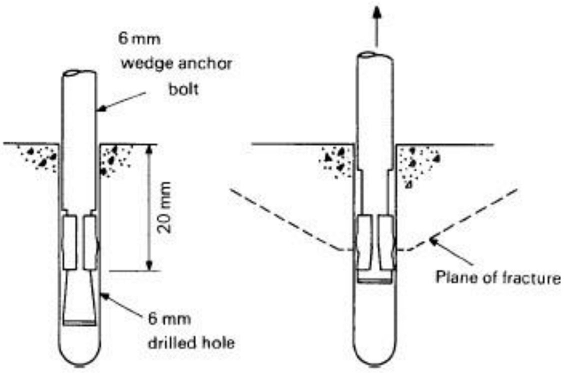


Figure 2.28: Schematic cut view of internal fracture test (Bungey et al. 2006).

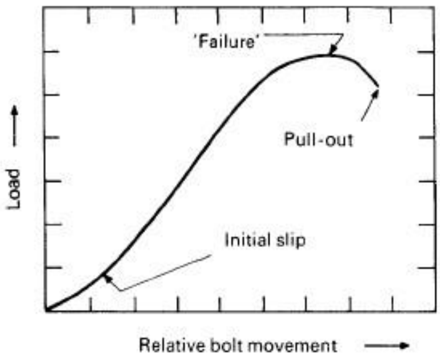


Figure 2.29: Peak load of the load versus movement (Bungey et al. 2006).

Figure 2.30: Pull force testing apparatus (Bungey et al. 2006)

2.1.2.3.2 Principle and theory of the method

During the test, failure occurs by cracking after the failure load is reached. The average observed failure length is 17 mm and 78 degree of half angle which is greater than the angle of the probable friction of concrete of 37 degree. That is because the failure consists of sliding and separation (Bungey et al. 2006).

2.1.2.3.3 Factors affecting the results

The test has high variability due to the localized nature of the test, the imprecise load transfer mechanism and the method of drilling (Bungey et al. 2006).

The size and type of aggregates affects the results to a large extent. The torquemeter force decreases for natural aggregates and there is difference in result with type of aggregate for torquemeter method. However, the direct pull force method gives closely related results for different aggregates. *Figure 2.31* illustrates the difference in the correlation curves of the two loading methods. (Bungey et al. 2006).

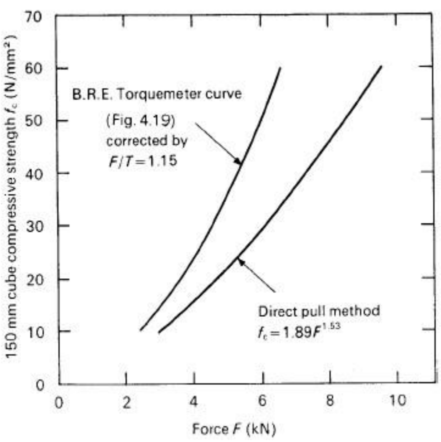


Figure 2.31: Correlation curves of the loading methods (Bungey et al. 2006)

2.1.2.3.4 Advantages and limitations

This method's results are affected by few variables. Further, the main advantage of this method is that a general calibration curve can be employed for natural aggregates only related to the specific method of application of load (Bungey et al. 2006).

The moisture content and the age of concrete have no effect on results. Carbonation can be neglected in most circumstances except when the depth of carbonation reaches the depth of test. In addition, the results are not affected by the change in composition of the concrete for natural aggregates. The accuracy of the results is comparable to small cores. The method is suitable for slender members with one exposed end and it is a preferred method when the age and composition of concrete is unknown (Bungey et al. 2006).

However, there is high variability of results with aggregate size and it requires more specimens for calibration. The maximum aggregate size is limited to 20 mm because of the limitation regarding the depth of the application of the method (Bungey et al. 2006).

2.1.2.4 The Pull-out test

This mechanical method is not fully non-destructive, rather it is a semi-destructive method. However, this method is useful since it is a common practical method for structural assessment which provide information directly associated with the mechanical properties of the concrete. It is also conceivable that the pull- out method could be combined with other non-destructive method, or else be used alone (RILEM 2012).

The first tests took place in USA and USSR in 1930 and were not popular since they were not developed enough. Thirty years later, these methods began to be more practical used because of their development. Denmark and Canada were the two countries that pioneered this technique. These countries used different ways of loading and different attachment parts. Both cases were based on the same principle, where a cone of concrete wall was pulled out in order to translate the required force to the compressive strength by an empirical calibration curve (RILEM 2012).

The pull-out test consists of two approaches, namely: the cut- and pull-out test (The capo test) and the Lok test. The approach that is being used to evaluate the in-situ strength of concrete is called the 'The Capo test'. This method is used for situations that has not been pre planned for determination of the compressive strength (Bungey et al. 2006). The Lok test is applied for planned situations when the aim is to measure the strength of the concrete in a newly built construction (Andrzej et al. 2016).

2.1.2.4.1 Principal and theory

The major notion behind the Pull-out test is that precise assessment of the strength of concrete can be estimated on-site by means of the required force to pull-out a fixed insert (Malhorta & Carino 2004). The pull-out force correlates correctly to the compressive strength of concrete evaluated through cubes or cylinders in the laboratory (RILEM 2012). An insert made of metal is either installed inside the hardened concrete by drilling or cast inside the fresh concrete. When one aims to estimate the in-place strength for mature concrete, the metal insert is pulled out by a jack that reacts against the bearing ring which later result in the pull-out strength being ascertained (ASTM C900 – 19).

When the purpose is to measure the compressive strength of concrete in new constructions, the procedure is the same as for The Capo test, except that the inserts are directly cast inside the fresh concrete as illustrated below in *Figure 2.35* (Andrzej et al. 2016).

Furthermore, it is of importance to make sure that the dimensions and procedure of both tests fulfil the requirements according to *Figure 2.34* and *Figure 2.35*. To prevent yielding during the test, the metal insert must have sufficient yield strength and thickness. The distances should be fulfilled according to *Figure 2.34*. Generally, the diameter of the counter pressure ring is presumed to be approximately 55 mm. Finally, the force required to pull-out the insert is to be measured through the hydraulic pull machine. The hydraulic pull machine is reacting against a counter pressure ring when the inserts are being pulled (RILEM 2012).

During the testing, the concrete is concurrently subjected to both shear and tension, see *Figure 2.33.* The figure illustrates a free body subjected to pull- out force where the force is equal to P and resisted by the normal stresses (σ) and shear stresses (τ). The tensile stress is the normal stress which act perpendicular to the surface, whilst the shear stress acts in the parallel direction as shown in the figure. The vertical force that counteracts the applied pull-out force, is produced by multiplying the area (A) to the vertical stress components (Malhorta & Carino 2004).

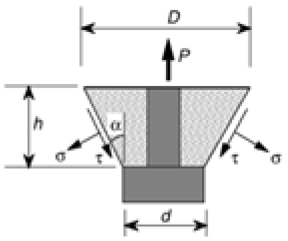


Figure 2.33: shows distribution of normal and shearing stress (Malhorta & Carino 2004).

D = bearing diameter

d = insert diameter

h = embedded depth (Malhorta & Carino 2004).

Further on, during the pull-out test, inner fractures occur, which is built up by a multi-phase process. This process consists of three separate stages with different observable fracture mechanisms. During the first phase, cracking starts to take place at the upper edge of the insert head because of the reached level of 30 to 40% of the ultimate load, have been achieved (see *Figure 2.36*). The ensuing cracks have a total length of approximately 15 to 20 mm counting from the edge of the insert’s head. During the second phase, the cracks pass from the top of the inserts head toward the base of the counter pressure ring. The crack pattern during the second phase is resembling to the vertical cracks arising within a cube or concrete cylinder when subjected to different compression tests (Bungey et al. 2006).

During the last and third phase when the load reaches the ultimate level, the last stage of rupture occurs. This results in shear/tensile cracks stretching to the edge of the counter pressure ring starting from the outer edge of the insert head. During the second phase of cracking, the rupture occurs in proportion to the load, therefore the pull-out force is proportionate to the compressive strength of concrete (RILEM 2012).

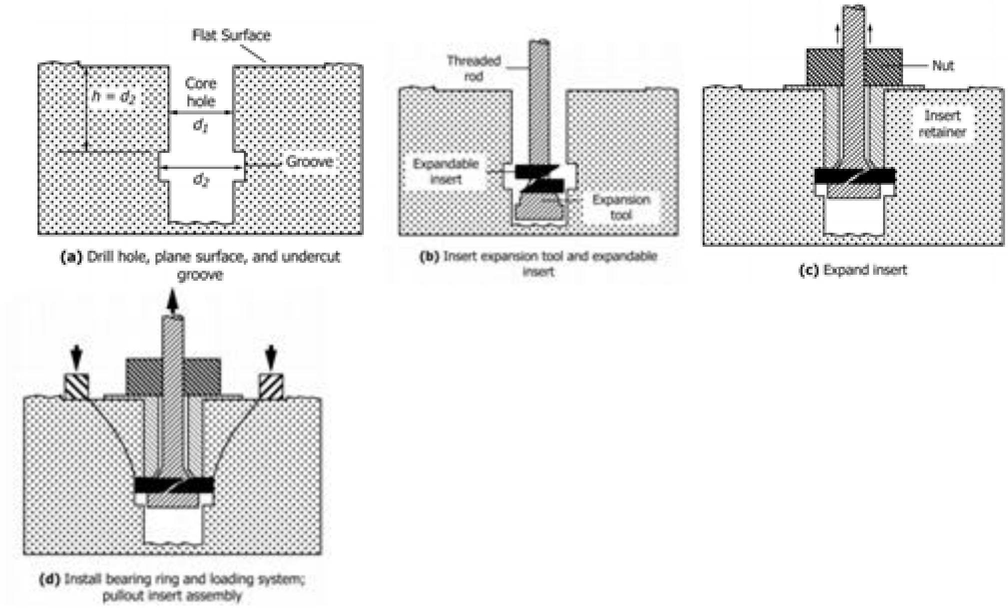


Figure 2.34: Schematic procedure of post-installed inserts

(ASTM C900 – 19)

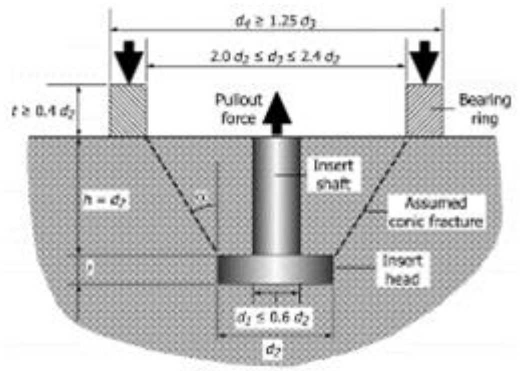


Figure 2.35: Schematic procedure of pre-installed inserts

(ASTM C900 – 19)

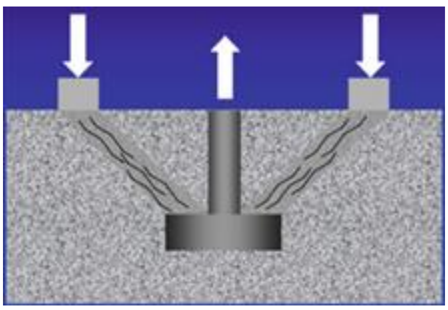


Figure 2.36: Fractures occuring due to state of stress

(Rilem 2012)

As mentioned above, the concept of the pull-out test is that the compressive strength of concrete is associated to the maximum pull-out force required to apply to the insert before the failure of the concrete (Malhorta & Carino 2004). There are two main categories for this kind of test namely: one where the insert is placed by drilling into the concrete (The Capo test) and the other where the insert is cast inside the concrete (The Lok test) (RILEM 2012). The main difference between these two is the pre- and post-planning of the installed inserts.

As written directly above, one of the procedures for determining the strength of the concrete is through installed insert by drilling. This technique is mainly used for determining the compressive strength in an existing structure where the determination of strength is accomplished on the site. This method is applicable during following cases:

- When the need of evaluation and verification of the in-place strength is crucial due to failure of the specimen.
- During technical surveys, when there is a need for assessing the proper and actual compressive strength of the material.
- During quality control.
- When one wants to test the remaining strength of the concrete before additional loading (RILEM 2012).

2.1.2.4.2 Method behind The Capo test

In situations where testing cannot be pre-planned, a development of the pull-out with an expanding ring was introduced into an undercut groove (Bungey et al. 2006). The fundamental geometry of each insert is similar where each insert has a disc with a diameter of 25 mm that is connected to a stem which places the disc 25 - 30 mm below the surface of the concrete, see *Figure 2.37* (RILEM 2012).

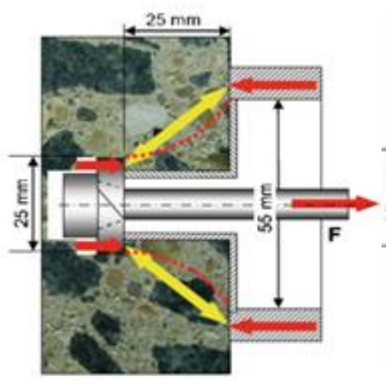
The insert is divided into two separate strength classes: 0 to 50 kN and 0 to 110 kN. The major disparity between them is the disc thickness where a diameter of 8 mm is applied for the 0-50 kN insert and the 16 mm in diameter is categorized for the 0 – 110 kN (RILEM 2012).

The first stage of the Capo test comprises of drilling a hole with diameter of 18 mm and at a depth of 45 mm. After that, a 25 mm diameter deep groove is cut at a 25 mm depth by applying a portable reaming device. Subsequently, the expanding ring is placed and it is expanded in to the groove by a pull bolt assembly (see *Figure 2.37*). The estimation of the cube compressive strength is being achieved through the empirical correlation curve (RILEM 2012).

2.1.2.4.3 Method behind The Lok test

The test is based on the same principle as the Capo test, where the tensile force is measured by pulling out an insert. But, in this case, the metal insert has been cast inside the concrete. To accomplish the test procedure, a manually operated jack is used to apply the load. This instrument bears against the surface of the concrete through a reaction ring with an internal diameter of 55 mm (Bishr 1990; RILEM 2012).

The inserts could either be attached to a plastic buoyancy cup that floats on the surface of the concrete (for slabs) or it could directly be attached to the formwork. The main geometry of the inserts is the same as for The Capo test (RILEM 2012) and they are illustrated in *Figure 2.39* and *Figure 2.40.*



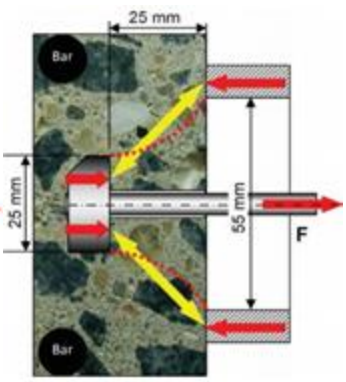
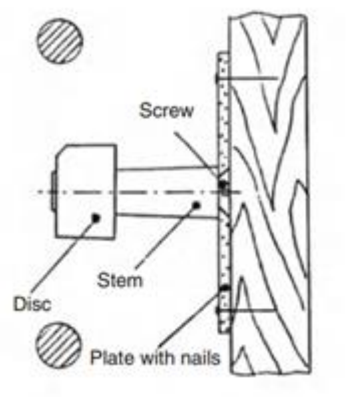


Figure 2.37: The Capo test Figure 2.38: The Lok test

(Rilem 2012) (Andrzej et al. 2016)



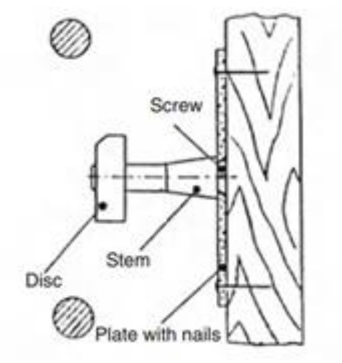


Figure 2.39: 0-110kN Figure 2.40: 0-50 kN

(Rilem 2012)

2.1.2.4.4 Factors affecting test result

The roughness of aggregate can yield unreliable test results (Schabowicz et al. 2018). Furthermore, the factors that affect the correlations and results are the size of the aggregate, temperature, relative humidity and the presence of reinforcement. Aggregates larger than 40 mm and lightweight aggregates affect the correlations. The correlation should therefore be developed and used for the specific concrete that is being evaluated. The presence of reinforcement causes inaccurate results and therefore bars must be avoided in the test zone (Malhorta & Carino 2004; Andrzej et al. 2016).

2.1.2.4.5 Standards and Correlations

Many years of experience achieved globally implies that one universal correlation is valid for all normal densities of concrete. However, this correlation is invalid for concrete consisting of maximum aggregate size larger than 40 mm and lightweight aggregate (RILEM 2012).

The standard EN 12504-3 is applicable as guidance about the procedures of testing and interpretation of the results (EN 12504-3).

From a statistical point of view, the 95% confidence limit for the compressive strength is around $\pm$ 20 % of the mean value performed on four tests (Andrzej et al. 2016). A standard European correlation curve is presented in *Figure 2.42*

The pull-out strength is calculated according to equation 2.6:

$$f_{\mathrm{p}} = \frac{\mathrm{F}}{\mathrm{A}} \qquad \text{where:} \qquad (2.6)$$

f_p = pull-out strength (MPa)

F = pull-out force (N)

A = surface area (mm^2); Area is calculated according to equation (2.7):

$$\pi\left((\mathrm{d}_2 + \mathrm{d}_1)\mathrm{h}^2 + (\mathrm{d}_2 - \mathrm{d}_1)^2\right)^{0,5}) \qquad \text{where:} \qquad (2.7)$$

d_1 = diameter of the head of pull-out insert (25mm)

d_2 = inner diameter of bearing ring (55mm)

h = distance from the pull-out insert head to the concrete surface (EN 12504-3).

There are two equations (equation 2.8 & 2.9) that can be assumed as appropriate approximation for the two curves that relate to the cube strength in *Figure 2.41*. These are:

$$F_{\mathrm{c,cube}} = 1{,}41\mathrm{P} - 2{,}82 \qquad \text{for strength under 50 MPa} \qquad (2.8)$$

$$F_{\mathrm{c,cube}} = 1{,}59\mathrm{P} - 9{,}52 \qquad \text{for strength over 50 MPa} \qquad (2.9)$$

(RILEM 2012)

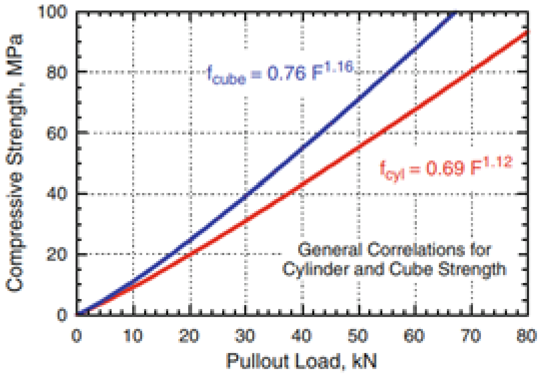


Figure 2.41: General correlation curves for both cylinder and cube (RILEM 2012).

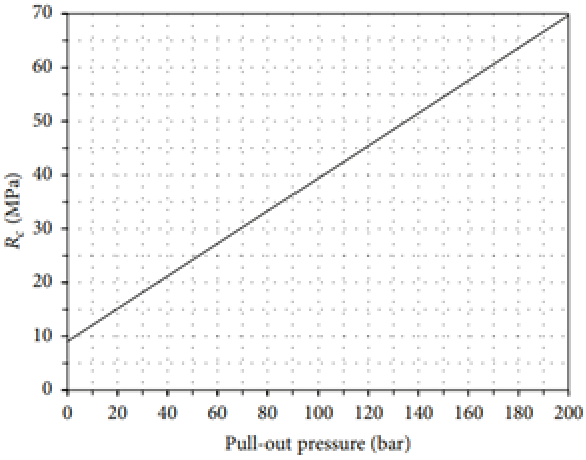


Figure 2.42: European standard correlation curve for pull-out (Bovio et al. 2014).

2.1.2.4.6 Advantages and limitations

The Pull-out test is a direct method meaning that the test result is available immediately after testing. The entire testing operation, including drilling, does not have to be pre planned for the Capo type of test. The test results have decent correlation with compressive strength. The moisture condition and maturity have negligible effects on the measurements (Bungey et al. 2006). Additionally, the factors that do not affect the correlations are the water to cement ratio, cement type, air entrainment, curing conditions and rigidity of the member (Andrzej et al. 2016).

There are however several disadvantages with the method. Cracks occur due to the state of stress after pulling out the insert which cause damage to the surface of the concrete. The equipment costs are high and the operation of applying the method is complex (Bungey et al. 2006).

The main limitation of the method is the temperature and relative humidity. According to EN 12504-3, the method could not be applied on frozen concrete. Another limitation is related to the post installed inserts that fail during tensile stress states (Bovio et al. 2014).

2.1.2.5 The Pull-off test

The semi-destructive methods in general consist of destructive forces causing damage that needs only superficial repair work. Further on, the structural strength of the member during the test is not affected. In non-destructive tests a parameter that need to be correlated with compressive strength test results is measured. However, semi destructive test results are considered more trustworthy than the non-destructive methods since a strength parameter is directly measured. Additionally, the semi-destructive methods are less expensive than the destructive ones (Naderi 2006).

The pull-off test is a mechanical, semi-destructive and direct method. It was established during 1970 and was developed in England with the purpose to ascertain the strength of the concrete material to evaluate the beams with high alumina cement (Pereira et al. 2012). This mechanical test procedure is based on the principle to measure the required tensile force to pull a metal disk from the surface of the concrete (Malhorta & Carino 2004). The requisite tensile force is associated with the compressive strength of concrete (Pereira et al. 2012).

The metal disk is adhered to the concrete surface with a two-part epoxy system. There are two ways for conducting the test procedure namely: one where the disk is in direct contact to the surface and the other is when partial coring is carried out. The partial coring is executed to a suitable depth to avoid the concrete that is exposed to carbonation or other deteriorated surface conditions (Malhorta & Carino 2004).

One way the test is applied, is by utilizing a portable hydraulic equipment (RILEM 2012). An additional way for measuring the pull-out is by using the ‘Limpet’. The ‘Limpet’ is operated manually with load applied axially. The tripod apparatus, also called the hydraulic apparatus is used by applying the load hydraulically or mechanically (Bungey et al. 2006). The equipment is illustrated below in *Figure 2.43* and *Figure 2.44.*

Figure 2.43: ‘Limpet’

(Bungey et al. 2006).

Figure 2.44: Hydraulic

(Bungey et al. 2006).

2.1.2.5.1 Principle and theory

Difficulties may appear regarding the adherence between concrete and the metal disc due to environmental conditions such as dampness (Bungey et al. 2006). Due to the adherence problem, the results obtained may become rejected. Hence, six discs should be tested to assess the compressive strength (Malhorta & Carino 2004).

The failure of the disc depends on the stiffness of the disc, the ratio between the thickness and diameter and the material that the disc is made of. The ratio is illustrated in *Figure 2.46* and it can be seen that the steel reaches maximum failure load when the thickness of the disk is 40% of the diameter in comparison to aluminum where a thickness is 60% of the diameter to reach failure (Bungey et al. 2006).

The nominal tensile strength is calculated on the foundation of the diameter of the metal disc. Subsequently, correlation can be developed between the tensile strength and compressive strength for the appropriate concrete. The correlation for the two methods is different. That is because the required pull off force is less for partial coring method (Bungey et al. 2006).

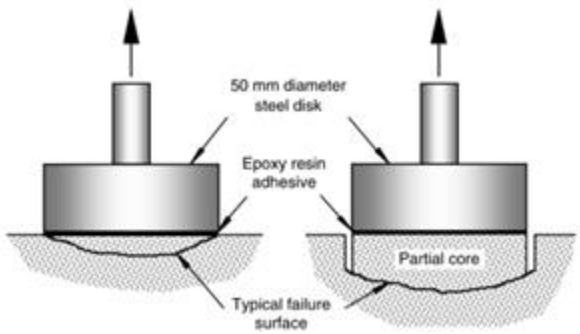


Figure 2.45: illustrates the two methods that can be applied.

(Malhorta & Carino 2004).

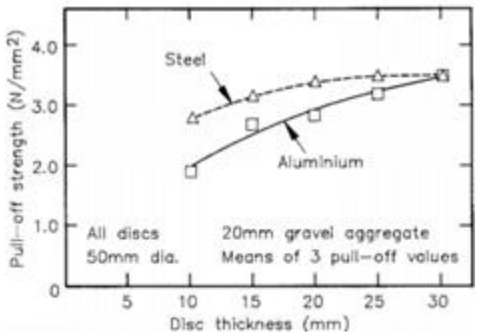


Figure 2.46: Illustrates the impact between type and thickness of disc

(Bungey et al. 2006).

2.1.2.5.2 Method for testing

The first step for the procedure is to get rid of laitance caused by cement and water on the concrete surface by scratching it off with a wire brush. Afterwards, the metal disk and the bare surface of the concrete are then lubricated to make sure decent bonding of the adhesive (Malhorta & Carino 2004).

The adhesive contains of a two- part epoxy system. This epoxy system is later dispersed over the disk for making it possible to adhere the disc onto the concrete surface. Excessive adhesive that has been pinched out during the procedure, ought to be removed before it sets. The environmental conditions and the type of epoxy alter the curing time of the adhesive. In most of the cases, the curing time take no more than 24 h. When the adhesive has reached a sufficient level of curing, the metal disc is pulled-off from the surface of the concrete (Malhorta & Carino 2004).

The Limpet is used for applying and recording the tensile force at the same time. The apparatus applies a force of 10 kN capacity at a speed of 6 kN/minute all through a screwed rod attached into the metal disc. The nominal pull-off strength is measured from the recorded tensile force that has been applied on the disc (with a diameter of approximately 50 mm) through the threaded rod. An established empirical correlation chart is used in order to convert the pull-off strength into a cylinder or cube compressive strength (Bungey et al. 2006). A typical correlation chart is shown in *Figure 2.47.*

2.1.2.5.3 Factors affecting test results

The location of reinforcement, the material of the disc and the speed of load affect the variability of the test results. During the test performance, it is vital to make sure that the metal disc is not bonded to the area of reinforcement. A reinforcement locator can be used to trace the reinforcement's position (Pereira et al. 2012).

Furthermore, the pull-off test only measures a surface layer of 5 mm thick concrete and in the case of partial coring the concrete property is measured up to the depth of partial coring. (Malhorta & Carino 2004).

At last, the factor that has the greatest impact on the correlation is the type of aggregate. Different types of lightweight aggregates affect the results and therefore separate correlations need to be used for lightweight aggregates and natural aggregates. The authors claim that the pull-off values for natural aggregates are lower than the pull-off values of the lightweight aggregates. *Figure 2.48* illustrates different correlations between different lightweight aggregates (Malhorta & Carino 2004).

2.1.2.5.4 Standards & Correlations

The Swedish standards do not mention anything about the pull-off method and the American standards do not discuss the correlation between the pull-off test and the compressive strength, rather it points out that: "This test method determines the tensile strength of concrete near to the prepared surface, which can be used as an indicator of the adequacy of surface preparation before applying a repair or an overlay material"- ASTM C1583/C1583M. Further on, it states the test procedure for how to measure the required tensile strength for pulling the bonded metal disc from the surface of the concrete.

According to Bungey et al. 2006, a 95 % confidence limit on compressive strength for six discs, resulted in a deviation of $\pm$ 15%

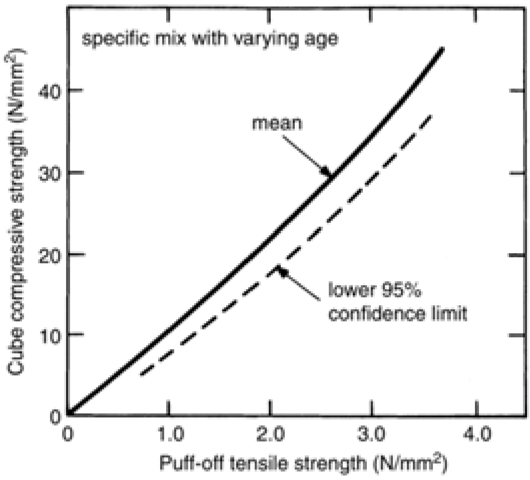


Figure 2.47: Illustrates an example for correlation between concrete and pull-off strength (Malhorta & Carino 2004).

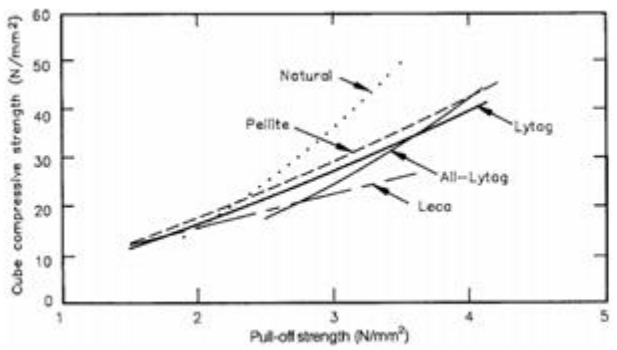


Figure 4.30 Typical strength correlations for lightweight aggregates (based on ref. 34).

Fig 2.48: Illustrates strength correlation for lightweight aggregates

(Bungey et al. 2006)

2.1.2.5.5 Advantages and limitations

The advantages with the method are the simplicity and time efficiency regarding the process for preparing the bonding between the concrete surface and the metal disc. This process does not take more than fifteen minutes. Another benefit is that the destruction caused on the surface does not cause any severe structural damage (Malhorta & Carino 2004).

The simplicity and rapidity with this method contribute to benefits for the user. The repeatability consisting of performing the test for six different metallic discs increase the reliability of the applied method. The test results are not affected by the aggregate type and size, compressive stress, curing, air entrainment and age of concrete (Bungey et al. 2006).

The required curing time for the adherence is a limitation that occur during this process. During most cases, the bonding of the disc takes place for one day and then the test will be performed during the upcoming day. There is, nevertheless, a prospective chance that the adhesive will fail due to unfavorable environmental conditions. It is, therefore, important to ensure proper surface preparation before bonding. Also, as mentioned earlier, another way to avoid this dilemma is by performing this test for six different metallic discs (Malhorta & Carino 2004). Still, thanks to recent accessible adhesive, the bonding during wet conditions could be made adequate. This has resulted in the pull-off test being able to be performed on concrete at an early age (RILEM 2012).

2.1.3 Destructive methods

2.1.3.1 Drilling cores

The compressive strength is one of the vital properties that is required for assessment of existing structures. The determination of the precise in-situ compressive strength is achieved by conducting destructive tests. The destructive method involves taking drilled core samples from the structural members that are to be investigated to test them in the laboratory (Ergün & Kürklü 2012).

The drilled core method is an established direct method of testing for evaluating the compressive strength of concrete. The method enables ocular assessment of the inner regions of the structural elements. This method also enables other physical properties to be measured, such as water absorption, indirect tensile strength, density and differential movements caused by for example alkali-aggregate reactions. Additionally, the drilled cores are commonly used as samples for chemical investigation regarding the strength properties of concrete (Bungey et al. 2006).

Further on, the quality of the concrete structure is identified by the compressive strength and the compressive strength is the measure of compressive force resistance per unit area. Meaning that a higher quality of the concrete has high compressive strength. The core drill method has become a preferred test for evaluating the accurate strength of the concrete since it is the most reliable method. Unfortunately, this method has disadvantages and takes a lot of time to ensure that the test would not induce damage on different parts of the structural elements (Sitorus & Jaya 2020).

According to some literature the orientation of the drill core with respect to placement could affect results because of anisotropic property of concrete. However, this view is not supported by other studies (Carroll et al. 2016). The drilling, preparation of drilled cores, the compressive strength tests and the interpretation of the results must be accomplished according to the following European standards:

- EN 12504-1:2019
- EN 12390–3:2019
- EN 13791:2019

2.1.3.1.1 Principle and theory

There are factors that influence the choice of the location of the drilling of the cores in a structure. The drilling of cores from the structural members must be performed by considering the stress distributions and strength variations of the structure. Meaning that the tests should be taken at specific non-critical points where there is no presence of reinforcement and where the strength of the structure is low (Bungey et al. 2006).

According to EN 12504-1:2019 the cores should be drilled at points where there is no reinforcement and away from the edges or joints. Furthermore, the drilling must be performed perpendicular to the surface in order not to damage the cores. If the drilled core contains any reinforcement bars, then it must be rejected and replaced by another one. (EN 12504-1:2019).

Before drilling one must decide the length of the cores and to do that several aspects need to be considered:

- The diameter of the core
- The potential procedure for adjustment
- Whether the comparison is to be made with cylinder or cube strength (EN 12504-1:2019).

After the drilling one must ensure that the orientation and the location of the core has been marked and recorded. If there are several specimens cut from one core, then the location and orientation of the specimens with regard to that specific core must be marked. Finally, the core specimens must be surface dried by means of a paper towel or dry cloth and subsequently put in a sealed polythene bag to prevent moisture exchange with environment (EN 12504-1:2019).

A rotary cutting tool is used for drilling the cores as illustrated in *Figure 2.49.* This tool is a heavy and portable equipment which must be braced and supported against the concrete to avoid unnecessary movements resulting in broken or distorted cores. This technique also necessitates uniformity of pressure; therefore, it must be managed by a skilled operator (Bungey et al. 2006).

It is critical to make sure that the drilled cores are examined carefully in terms of controlling if there is any excessive reinforcement, voids or if the length of the core is insufficient for testing, otherwise extra specimens must be drilled from these specific locations (Bungey et al. 2006).

Finally, to identify each core, they must be separately clearly labelled. Immediately after cutting, photographs of the cores should be taken. These photographs will be useful for future reference such as confirming different type of features that was observed during the inspection (Bungey et al. 2006). An example of a photograph is illustrated in *Figure 2.50.*

Figure 2.49: A rotary cutting tool for drilling the cores. Figure 2.50: Example of a labelled drilled core

(Bungey et al. 2006). (Bungey et al. 2006).

2.1.3.1.2 Method of testing

After the drilling, the extracted cores are then carefully examined and prepared in terms of cutting their ends and grinding them (Milius et al. 2015). Afterwards, these cubes or cylinders are loaded until failure in the laboratory by a compression testing machine. The maximum load required for reaching the failure is then recorded and the compressive strength of the concrete is calculated (EN 12390:3).

Before the compression test, one must consider the length and diameters of the cores. There are two limited ratios for the length/diameter:

- 1,0 if the strength results are compared to cube strength.
- 2,0 if the strength results are compared to the cylinder strength (EN12504-1:2019).

The preparation before the compression test does also consists of wiping the surfaces of the cores from extraneous material and any loose grit since the surfaces of the specimens must be clean when they are in contact with the platens of the test machine. The cores are then positioned in a way that allow the load to be applied perpendicular to the direction of the casting (EN 12390-3:2019).

When completing the preparation process the loading starts to take place. The starting point is to select a constant rate of loading ranging within 0,6 + 0,2 MPa/s. After the applied initial load to the specimen, one must make sure not to exceed the load with shock, rather one must increase it at a selected rate of + 10% until no greater load can be applied. Finally, the recorded maximum load should be expressed in kN and calculated according to equation (2.10) (EN 12390-3:2019).

$$f_c = \frac{F}{A_C} \qquad \text{where:} \qquad (2.10)$$

f_c = the compressive strength in MPa

F = the maximum load at failure in N

A_c = the cross-sectional area of the core on which the compressive force acts

The compressive strength is in the end indicated to nearest 0,1 MPa (EN 12390-3:2019).

2.1.3.1.3 Factors affecting test results

There are many parameters affecting the compressive strength of the cores such as: core diameter, magnitude of core compressive strength, coring orientation, the ratio between core diameter and length, presence of reinforcement and moisture condition in the core. To obtain reasonable compressive strength of the cores, one must consider all the mentioned factors by applying correction factors (Ergün & Kürklü 2012).

The location of the concrete in a structure affects the strength of the concrete, meaning that the concrete at the top of the structure element is weaker than the concrete at the bottom of the same element. Likewise, the core strength is affected by the core orientation where the strength of the concrete is lower parallel to horizontal plane. Therefore, it is of importance to consider these factors during the planning of the drilling process (Sitorus & Jaya 2020).

Large aggregate particles in a core affect the drilling process since the cutting of core can result in loosened aggregate. Also, larger sizes of aggregates will also cause interfacial transition zone (ITZ). The ITZ cause weaker parts within the core itself. Generally, larger size of aggregate and high w/c ration in the ITZ result in more permeable and weaker concrete (Carroll et al. 2016). Additionally, if the maximum aggregate size increases, then the compressive strength of concrete core will decrease (Ergün & Kürklü 2012).

The moisture condition in a specimen during the compression test, affects the strength of it. There is however, no general procedure that will ensure that the moisture condition in the specimen during the time of the drilling will be the same during the compression test. The only way to prevent moisture change before the testing of the specimen is to put the cores in polyethene bags as mentioned above (Sitorus & Jaya 2020).

Another factor affecting the compressive strength of the cores is the diameter of the core. Cores with smaller diameters have lower compressive strength and cores with larger diameters have higher compressive strength (Ergün & Kürklü 2012).

Finally, the ratio between the length and the diameter of the core alters the compressive strength of the cores, therefore following correction factors (see *Table 2.2*) should be multiplied to the final acquired results:

Table 2.2: Correction factors

Cores	Ratio (l/d)	Correction factor
1	2.00	1.00
2	1.75	0.98
3	1.50	0.96
4	1.25	0.93
5	1.00	0.87

(Sitorus & Jaya 2020).

2.1.3.1.4 Advantages and limitations

The drilling core method is a direct method which immediately give the compressive strength of the specimens tested in the laboratory. There are however several disadvantages and limitations with this method (Alwash 2017).

The main limitation with drilling cores is the core locations. One must carefully consider where to drill in order to not weaken the structure. Further on, the locations have to be accessible in order to facilitate the movement of the apparatus and the operation of drilling. Moreover, drilling consists of complex processes including, verifying the perpendicularly, setting up the machine and drilling the core and each step must be performed according to the requirements stated in the standards. Also, these procedures need to be accomplished by a skilled operator so that a representative and undamaged in-situ core sample is obtained (Alwash 2017).

The procedures according to European standard require a large number of cores, which can incur high costs (Alwash 2017).

2.1.3.1.5 Estimating of in-situ compressive strength

There are clauses in the European standard, EN13791, for determination and requirements for the in-situ assessment. Clause 8.1 in EN13791 present *"Estimation of compressive strength for structural assessment of an existing structure": Based only on core test data.* This section presents how many valid tests should be performed for different sizes of the specimens. In accordance with this clause:

- 8 valid test results should be obtained for core greater than 75 mm.
- 12 valid test results for should be obtained for cores greater than 50 mm.

A more detailed description of the interpretation and estimation of the in-situ compressive strength of concrete will be presented in section 2.2

2.1.4 Summary of advantages, disadvantages and limitations for all the methods

Table 2.3: pros and cons for each method

Method	Advantages	Disadvantages	Limitations
UPV	-No damage -Quick and easy to perform. -Low cost -Repeatability	-Many factors affect the test -Increasing w/c-ratio→increase pulse velocity -Sensitive to different environmental conditions (for eg. Waves)	-Reliability of absolute strength correlations: poor -Two opposite surfaces should (for more accurate results) be accessible. -Perform the test where there is no reinforcement. -Temperature variations
Pull-out	-Does not have to be pre planned. -Good correlation with compressive strength -Moisture condition and maturity have negligible effects. -The entire testing operation may be completed in about ten minutes. -Direct method	-Occurrence of cracks -Size and type of aggregates affect the results. -Damaged area after the testing. -The post-installed inserts fail during tensile stress states. -Equipment costs are high. -Operational complexity	-Cannot be applied on frozen concrete. -Problems may arise from the presence of reinforcement within the test zone, and bars must be avoided within the failure region-
Pull-off	-Quick and simply to perform→ take no more than 15 min. -The bonding during wet conditions could still be enabled. -Repeatability -Not marginally affected by age, aggregate type and size, air-entrainment,	-Failure of adhesive because of environmental conditions -Factors affecting the results: material of the disc, the speed of load, the position and orientation of the aggregate onto the disc. -Greatest effect on this relationship is the type of coarse	-Make sure that the metal disc is **not** bonded in the area of reinforcement →affect results. -The required curing time for the adherence, usually takes one day. -Must ensure proper surface preparation -The pull-off test only account and measure a surface layer of 5mm thick concrete -Stiffness of the disc

	compressive stress and curing	aggregate used in the concrete.	-Separate correlations are required for different types of lightweight aggregates -More suitable for small section members
Rebound hammer	-Reliable and quick for measuring uniformity -Gives consistently reproducible result than any other method. -Results sensitive to change in quality, inadequate mixing & segregation. -More relation with mechanical properties than any other method	-Least reliable in compressive strength result -Cannot measure property at depth into concrete. -The method even with calibration is not reliable alone.	-Cannot be used in frozen surfaces -Limited by slenderness. EN 12504-2 recommends size should be 100mm
Windsor probe	-Is not affected by operator's skill. -Variability of results is low. -Does not require power source -One of the best combined ratings in terms of reliability, simplicity, accuracy and economy -Correlation affected by small number of variables -Gives accuracy of small diameter cores	-Requires safety cautions -More complex correlation because of two power levels -Surface may need patching -Old concrete may cause overestimation	-Minimum edge distance for avoidance of cracking of concrete, minimum distance between probe locations and minimum size of samples are required. -Distance of reinforcement specially less than 100 mm can have effect on depth of penetration
Windsor pin	-Simple, low cost, speedy and with low damage -No effect of temperature and moisture content	-Type of aggregate and type of mix affect results -Carbonation affects result	-Applicable to limited range of strength -Limited for freshly hardened concrete and mortar

	-No effect of reinforcement depth	-Damage of pins on hard surface	
BO method	-Quick and uncomplicated -Good accuracy and reproducibility of results -Good correlation with compressive strength -Result unaffected by surface conditions, temperature, shrinkage	-Damages surface of concrete	-Max aggregate size 20mm and min member size 100mm
Internal fracture test	-Accuracy comparable to small cores -Method suitable for slender members -Method suited when concrete composition and age is unknown	-High variability with aggregates size -Requires more core specimen for calibration -Load application method and rate affect results	-Max aggregate size 20mm
Drilling cores	-Direct method	-Destructive method -High costs -Operational complexity	-Orientation of the core -Requires location free from cracks and high stresses

2.1.5 The influence of the factors that affect the readings.

Table 2.4: factors affecting the readings.

Method	Rebound hammer	Windsorprobe	Windsor Pin	Ultrasonic pulse velocity	Pull-out	Pull-off
Parameter	**RN**	**EPL**	**PD**	**UPV reading**	**Force**	**Force**
Destructiveness of test	ND	SD	SD	ND	SD	SD
Concrete composition						
Aggregates-content	*	*	*	**	o	*
size	*	*	*	*	*	**
Origin (density, hardness)	**	**	**	**	o	o
Cement-content	o	o	o	o	o	o
type	*	o	o	*/o	o	o
Concrete related properties						
concrete age (hydration degree)	**	**	**	*	o	o
presence of rebars/reinforcement	*	*	o	*	*	**
presence of cracks and voids	**	o	o	**	**	**
Moisture content	**	*	*	**	o	o
Carbonation	**	*	**	o	o	o
Other parameters						
Thickness of tested elements	*	*	*	**	o-*	o-*
surface roughness (formwork type)	**	o	o	*	o	**
Temperature	o	o	o	o	**	*
Relative humidity	*	o	o	*	**	*
stress state	o	o	o	**	o	o

**-high influence

*-low influence

o-no influence

Non-destructive: ND

Semi-destructive: SD

EPL-exposed probe length
DP-depth of penetration

2.2 Interpretation of results

2.2.1 EN 13791:2007

The European standard, EN 13791:2007, is intended to be used for estimating the characteristic in-situ compressive strength of concrete in existing structures. The standard is applied to determine the characteristic compressive strength for old structures and for newly built structures. The compressive strength is considered as unknown for old structures whereas for new construction the purpose is to check conformity of the concrete. Our study covers only the case of old structures which is presented in clause 7 of the standard.

The procedures of the assessment of old structures presented in EN 13791:2007 include the use of drilling core tests and non-destructive testing methods. The non-destructive methods consist of ultrasonic pulse velocity, rebound hammer and pull-out.

According to this standard the assessment of in-situ compressive strength using only drilling cores is covered in clause 7 and clause 8.2 and 8.3 cover the indirect test methods.

2.2.1.1 Estimation of characteristic in -situ compressive strength with cores

There are several qualities to consider during determination of in-situ core strength:

- The testing of core with an equal length and diameter of 100 mm (where l/d = 1) has equivalent strength with cylinder with a diameter of 150 mm that has been cured and manufactured during same conditions.

- The testing of core with a length to diameter ratio equal to two where the diameter is 100 mm and not larger than 150 mm is equivalent in strength as a cylinder with a length of 300 mm and a diameter of 150 mm that has been cured and manufactured during same conditions.

- The established results from cores with diameters from 50 mm to 150 mm and with other length to diameter ratios, shall be adjusted by using conversion factors.

The number of drilled cores taken from one test region depend on the volume of the concrete however, a test region shall be based on at least three cores. This limitation is based on the cores with a diameter of at least 100 mm, if the diameter of cores is less than 100 mm then the number of cores must be increased.

Moreover, the in-situ characteristic compressive strength is evaluated applying either approach A or approach B. Approach A is used when at least 15 cores are obtainable and approach B is applied when there is availability of 3 to 14 cores.

<u>Approach A</u>: The in-situ characteristic strength of the test region is the lower of equation 2.11 and 2.12:

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{m(n),is}} - \mathrm{k}_2 \times \mathrm{s} \tag{2.11}$$

or

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{is,lowest}} + 4 \text{ where:} \tag{2.12}$$

s = standard deviation of test results or 2,0 N/mm^2, whichever is the highest value
k_2 = given in national provisions, if there is no value given then use k = 1,48

Approach B: The in-situ characteristic strength of the test region is the lower of equation 2.13 and 2.14:

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{m(n),is}} - k \qquad (2.13)$$

or

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{is,lowest}} + 4 \qquad (2.14)$$

The margin *k* varies with number of test results (n) presented in Table 3 in the standard

2.2.1.2 Estimation of characteristic in-situ compressive strength by indirect methods

These non-destructive or semi-destructive methods may be applied after calibration with core test according to following ways:

- In a combination of indirect methods
- Individually
- In a combination of indirect methods and direct methods

There are two ways for correlating the indirect test results with the cores depending on the number of cores. These two alternatives are presented below.

Alternative 1:

This alternative is based on direct correlation with cores. This method provides that at least 18 core test results and 18 indirect results are available, in order to obtain relationship between the result from indirect method and the in-situ compressive strength. The indirect tests must be performed at the same test regions as the drilled cores in order to correlate the results. Moreover, to establish the relationship between these two methods, one must determine a curve where the indirect test results are variables and the estimated in-situ compressive strength is a function of these variables.

The characteristic in-situ compressive strength $f_{is,\,I}$, is calculated for the specific concrete and the following conditions must be fulfilled:

- The assessment should be based on at least 15 test locations for each test region.
- The standard deviation must be the value calculated from the test results or equal to 3,0 N/mm^2, the highest value should be chosen.

Finally, the in-situ characteristic compressive strength is the lower value of equation 2.15 and 2.16:

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{m(n),is}} - 1{,}48 \times s \qquad (2.15)$$

or

$$f_{\mathrm{ck,is}} = \mathrm{f}_{\mathrm{is,lowest}} + 4 \tag{2.16}$$

Alternative 2:

This method is based on calibration with cores when there is a limited strength range, and an established relationship is used for this alternative. The established relationship is based on a basic curve which is calibrated by means of core tests. There are specific basic curves provided in this standard for the three indirect methods namely: ultrasonic pulse velocity tests, pull-out tests and rebound hammer tests.

The calibration of the test results is done by shifting the curve according to the relationship with core test results. This method is used for the population of normal concretes consisting of the same type of materials and produced under the same manufacturing process.

The test region is chosen from the same population of concrete and at least 9 pairs of results of indirect and core tests are required for the correlation. These test results are then used to calculate the value **Δf** (shift), that adjusts the results from the basic curve according to the core test outcomes. From the established relationship the estimation of in-situ compressive strength is accomplished, and the characteristic in-situ compressive strength can now be calculated. The basic curves for rebound hammer, ultrasonic pulse velocity and pull-out tests are presented in clause 8.3.3.

To sum up, the main difference between alternative one and two is the number of available core results and the curves that are being used. For alternative one, the curve is built up by correlating the indirect test results to the core results while alternative two demand the use of a basic curve due to the smaller number of cores.

2.2.2 EN 13791:2019

The EN 13791:2019 standard is a revision of the EN 13791:2007 European standard. The standard supersedes the previous one; however, the previous standard is still widely applied in Sweden. This standard has removed approaches A and B in the previous standard. The standard provides guidance and procedures in estimating the characteristic in-situ compressive strength based on EN 1990 and EN 1992-1-1. The hypothetical steps in determining in-situ the compressive strength according the standard is presented in *Figure 2.51* (EN 13791:2019).

Clauses 8 and 9 from the standard cover the procedures and steps to be followed for estimating the in-situ compressive strength of concrete. Clause 8 is the part devoted to estimation of compressive strength for structural assessment of an existing structure when there is no prior knowledge about the in-situ compressive strength. Clause 9 is out of scope of our thesis, since it covers the assessment for conformity of the concrete in case of doubt about the quality of the concrete (EN 13791:2019).

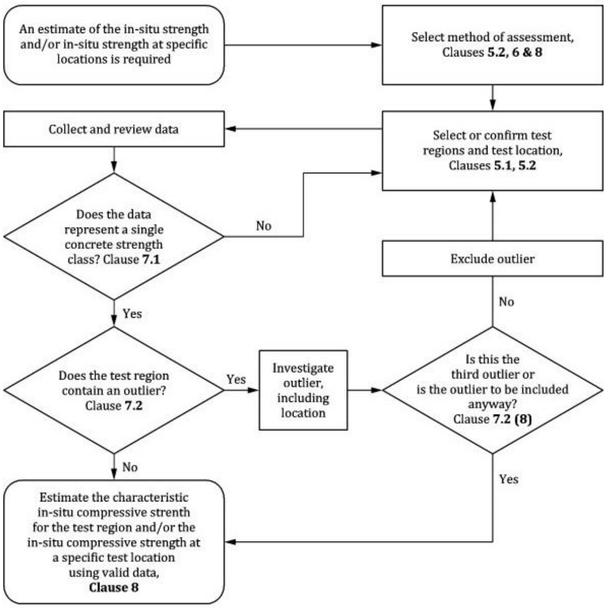


Figure 2.51: Flowchart of determination of in situ compressive strength (Clause 8)

2.2.2.1 Application and limitations of the method

The selection of the test locations should be carried out avoiding cracked areas, highly stressed or critical sections and reinforcement bars. The way to assign test results to test locations and the application of the various test methods in a test region are presented in Table 3 of the standard. The chosen number of tests should consider the volume of the concrete in a test region, the aim of the test and the expected accuracy of estimation (EN 13791:2019).

Before the estimation of the strength is considered, the data should be evaluated for statistical outliers and the test regions should be selected correctly. The test regions shall be formed by single elements or group of elements which are assessed to consist of the same quality of concrete. If the tests show that the test region is composed of two different classes of strength, the test data should be split into two separate data.

In cases where there is scarce data from the construction regarding the quality of concrete, other supplementary methods of classification can be adopted. In such cases, the use of NDT tests or engineering judgment is an alternative method to assess the quality of the concrete. The test results should be checked for outliers (EN 13791:2019).

It is recommended that engineering judgment and site-specific conditions must be considered for the application of the requirements set by the standard (EN 13791:2019).

The procedures of the core testing and conversion to in-situ strength results, are covered in EN 12504-1 and the determination of densities are described in EN 12390-7 (EN 13791:2019).

2.2.2.2 Estimation of the characteristic in-situ compressive strength using only cores

Scenario 1: For a minimum of 8 cores

Provided that the test results are valid: for a test region the number of cores required according the to the size of the cores are as follows:

- Minimum of 8 valid test results of in-situ compressive strength test results if the diameter of the cores is ≥ 75mm (test results should fulfill requirements set in Table 4 of the standard code).

- Minimum of 12 valid test results from single 50 mm diameter cores from concrete with maximum aggregate size of ≤ 16 mm (EN 13791:2019).

The steps of the estimation of in-situ characteristic compressive strength are:

1) Calculate the mean in-situ compressive strength $(f_{\mathrm{c,m(n)is}})$ and the standard deviation s of the test region from the valid test results of the in-situ compressive strength values$\left(f_{\mathrm{c,is}}\right)$

2) The characteristics in-situ compressive strength$\left(f_{\mathrm{ck,is}}\right)$ is the lower of

$$f_{\mathrm{ck,is}} = f_{\mathrm{c,m(n)is}} - k_n s \quad (2.21)$$

Where k_{n} is taken from table 6 of the standard code, or

$$f_{\mathrm{ck,is}} = f_{\mathrm{c,is,lowest}} + M \ , \quad (2.22)$$

Where M is based on the value of $f_{\mathrm{c,is,lowest}}$ and Table 7 in the standard. (EN 13791:2019)

Scenario 2: When the number of cores (n), fulfills $3 \leq n \leq 7$

Case 1: Small test regions when volume of concrete is 10 m^3 or less (Section 8.1(7))

If a test region comprises of one to three elements and the volume of concrete under investigation is less than 10 m^3, at least 3 cores of diameter ≥75mm with at least one core from each element should be taken. The in-situ compressive strength$\left(f_{\mathrm{c,is}}\right)$ is calculated and the lowest value is considered for structural assessment provided that the spread of the results is less than 15% (EN 13791:2019).

Case 2: Use of indirect testing with at least three core data (when volume of concrete is 30 m^3 or less (Section 8.3))

The test results are estimated using indirect test results without calibration. Indirect tests are made and the location with lower compressive strength are identified. Minimum of three cores of diameter ≥ 75 mm or an equivalent number of small cores are taken from the area with the lower indirect test results and the mean of the results is taken as the in-situ compressive strength$\left(f_{\mathrm{ck,is}}\right)$ provided the spread of results is less than 15% (EN 13791:2019).

2.2.2.3 Estimation of characteristic in-situ compressive strength using cores and indirect tests

At least 8 pairs of core test and indirect test results at the same locations are required to set up a correlation between the indirect tests and the in-situ compressive strength. The location of cores shall cover the extremes of the indirect tests as far as the execution of the test is possible. Regression equations can be set up from the relationship between the core tests and the in-situ compressive strength$\left(f_{\mathrm{c,is}}\right)$ results at the test locations. The established regression equation shall be employed for converting the indirect tests to equivalent regression values$\left(f_{\mathrm{c,is,reg}}\right)$, even in test locations where the core results are available.

For a selected test region, the first step in estimation of the characteristic in-situ compressive strength is by computing the mean in-situ compressive strength $\left(f_{\mathrm{c,m(m)is}}\right)$ given by equation 2.23 of *m* estimated strength values from the correlation curve. Then the overall standard deviation *s* of the test region given by equation 6 in the clause 8.2.2 of the standard is calculated. The effective degree of freedom is then calculated using equation 9 in the standard.

$$f_{\mathrm{c,m(m)is}} = \sum\left(f_{\mathrm{c,is,reg}}\right)/m \qquad (2.23)$$

The characteristic in-situ compressive strength of the test region is estimated by equations 2.21 and 2.22 by replacing n with the effective number of degrees of freedom n_{eff} according to Table 6 in the standard.

Where n is the number of pairs of the test results used to establish the correlation curve and m is the number of estimated strength values (EN 13791:2019).

The standard provides guidance for estimating in-situ compressive strength at a location since it is different from the estimated mean in-situ compressive strength of the test region. The section 8.2.3 in the standard covers the evaluation of in-situ compressive strength at a test location$\left(f_{\mathrm{c,is,est}}\right)$ valid only for linear correlation.

2.2.3 Assessment of the EN 13791:2007 and EN 13791:2019

The standards that are published yet are assessed to have difficulty of practical application because the methods require large number of cores and the methods are too conservative. (Rilem Technical Committee 249-ISC).

There is also shortcoming of the use of NDT in the practice because the standards require large number of cores. On the other hand, the clients in projects prefer to reduce the cost and the investigations being done are mainly combined NDT with small number of cores. Recent study by simulation methods, reports that for high quality NDT, four cores are sufficient and for medium quality NDT, five to nine cores are sufficient (Dyson et al. 2017). The upward shifting of the curve in EN 13791:2007 is doubtful because it lacks evidence (Broovsk 2009).

Although the assessment formulations are changed, there is no methodological differences between the standards in 2007 and 2019. In the new standard EN 13791:2019 approaches A and B are removed and more comprehensive guidance on the application procedures is included. New sections are added about how to define test locations and region. Additionally, the new standard provides guidance about initial assessment of data and planning of investigation for testing (EN 13791:2019).

The flow charts in *Figure 2.52-2.54* illustrate the difference in the provisions of the standards for estimating the characteristic in-situ strength of test regions.

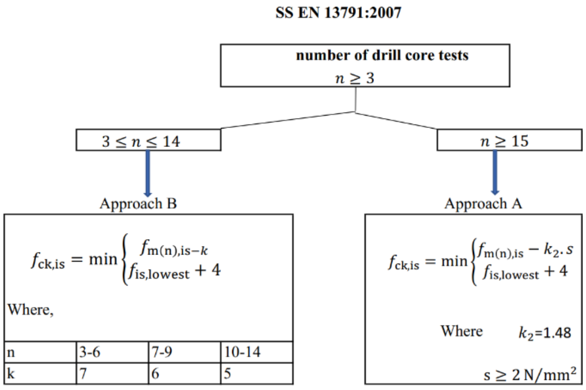


Figure 2.52: Flowchart of the procedures of estimation of in-situ compressive strength using only core drill tests (EN 13791:2007).

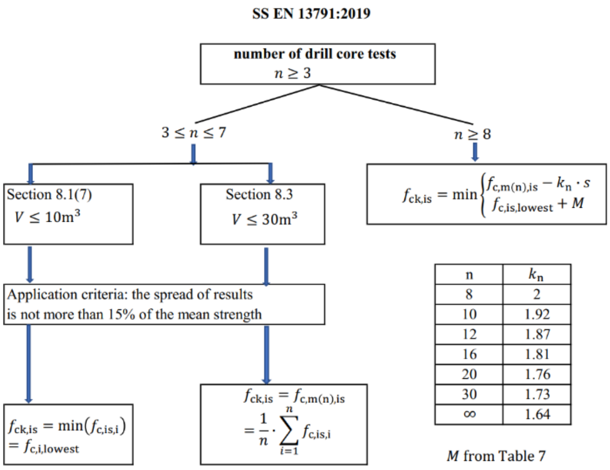


Figure 2.53: Flowchart of the procedures of estimation of in-situ compressive strength using only results from core drilling tests according to (EN 13791:2019).

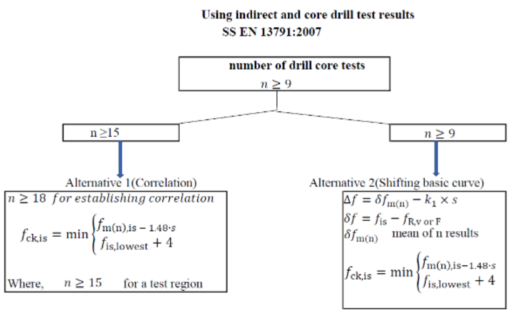


Figure 2.54: Flowchart of the procedures of estimation of characteristic strength of test region using indirect and core drill test results (EN 13791:2007).

SS EN 13791:2019

number of drill core tests
$n \geq 10$ (Correlation)

$$f_{\mathrm{ck,is}} = \min \begin{cases} f_{\mathrm{c,m(n),is}} - k_{\mathrm{n}} \cdot s \\ f_{\mathrm{c,is,lowest}} + M \end{cases}$$

n_{eff}+1	k_{n}
8	2
10	1.92
12	1.87
16	1.81
20	1.76
30	1.73
∞	1.64

M from Table 7
n_{eff} from Formula 9

Figure 2.55: Flowchart of procedures of the estimation of characteristic in-situ strength of existing structures using results of indirect tests and core drilling tests (EN13791:2019).

2.2.3.1 Assessment of EN 13791:2007

In a study carried out by Caspeele 2010 on simulated theoretical data, sometimes low spread of the estimated characteristic strength is observed in approach B. The reason is that the results depend only on the mean value and the parameter k is independent of the standard deviation. If the standard deviation is large, the computed probability of exceedance was 80-90%. On the other hand, the approach A results in lower probability of exceeding due to the constant k_2=1.48 which is a factor of the estimation equation (Weber 2019).

In another study of theoretical investigation of a set of samples, Knab and Sodeikat reported that The EN 13791:2007, approach B works well when the standard deviation is low. The corresponding maximum estimate for number of cores three to fifteen, became overestimated by 108 and 113%. Furthermore, the confidence also lies between 90 and 61% for three to ten cores. The confidence level drops to 36% as the approach is changed to A for fifteen cores (Weber 2019).

However, for large standard deviation the maximum estimate is overestimated by 145-217% for three to fifteen cores. Hence, the estimate becomes very high for use in structural calculations. When using approach, A, the estimation's reliability was low, at 52 percent for 15 samples. Approach B's reliability is still low, at 5 to 24 percent. Furthermore, an increase in standard deviation leads to an increase in the estimated characteristic strength (Weber 2019).

In a study by Holicky, the results of EN13791:2007 and EN 1990 were compared using stochastic analysis of a population of data. Approach B was used for samples with number of cores three to fifteen and for number of cores 15-30, approach A was utilized. The results from the EN 13791:2007 were larger than EN1990 for all simulated samples. The difference of estimation increases to maximum of 8 MPa when the samples size decreases (Weber 2019).

The application of approach B of EN 13791: 2007 on existing structures was studied for number of cores three to eight. The results showed a spread of 40% to 250% of the characteristic in-situ concrete compressive for three cores. For eight cores, the scatter that is reported is 65% to 200%. Furthermore, the findings show that increasing the sample size leads to a small reduction in spread. Additionally, as the total population's standard deviation rises, the characteristic compressive strength increases as well, and the spread is highest for three cores. Since the standard deviation covered by k is so small, high estimates are possible for high mean core test results (Weber 2019).

Therefore, the application of EN 13791:2007 is not recommended for the assessment of structural safety. The approach can lead to unsafe assessment of the characteristic strength. It is reported that approach B in EN 13791:2007 can result in an overestimation up to 40 MPa. Therefore, it is not recommended to use the approach for structural safety recalculations (Weber 2019).

A New modified method is developed that minimizes overestimation by limiting the maximum coefficient of variation. The method is developed from experimental findings which showed that the decrease in spread leads to low overestimation. The maximum overestimation of the new method known as DIN EN 13791: A20 2017-02 is up to 15 MPa whereas that of EN 13791:2007 is 40 MPa. The use of this method for small number of cores is advisable since it results in safe values, however the obtaining of high results is not ruled out (Weber 2019).

2.2.3.2 Assessment of EN 13791:2019

The EN 13791:2019 standard has changed the methodology of calculating the characteristic in-situ compressive strength. The statistical formulation of the calculations is now aligned with EN 1990. The equation utilized for computing the characteristic in-situ compressive strength is the 95% probability of t-statistic as presented in equation 2.24 (Harrison 2015).

$$f_{\mathrm{c,is,ck}} = f_{\mathrm{c,m(n)is}} - t_{0.05} s_{\mathrm{n}} \sqrt{\left(1 + \left({}^{1}/_{n}\right)\right)} \qquad (2.24)$$

In the current EN 13791:2019 and the coming EN 13791:2020 significant changes have been made to solve problem of accuracy for the case when the number of cores is between three and seven. The clause 8.1(7) of EN 13791:2019 demand requirement that the range of test results is a minimum of 15% of the mean of the test results ($R \leq f_{\mathrm{cm(n),is}}$). According to Rabea Sefrin, the statistical robustness of the criterion that limits the spread of the results is not supported by evidence, thus it is not reliable. The evaluation according 8.1(7) of EN 13791:2019 usually results in overestimation of results as is observed with approach B of the predecessor standard (Sefrin et al. 2020).

Therefore, the conclusion is that the spread is not robust statistical criteria because it does not enable the sorting out of values with large difference from the mean. Hence, it is recommended to use a robust estimator such as a minimum value criterion of coefficient of variation that improves the accuracy of estimation. The use of evaluation according to the national Germany standard DIN EN 13791/A20:2017 gives results that are more accurate than the other standard versions. Hence, the results can be used for calculating structural safety of structures (Sefrin et al. 2020).

2.2.4 TDOK 2013 0267

The compressive strength f_{ck} should be chosen in accordance with the concrete strength class ,*K-value* marked in the specific drawing related to the bridge. The characteristic values for the compressive and tensile strengths are presented in Table 1-5 in the standard (TDOK 2013:0267).

There are, however, specific guidelines for how to determine the compressive strength for the bridges built during different years and how to evaluate the compressive strength of the drilled samples in agreement to the given K- values. The following guidelines for determination of compressive strength will be presented below (TDOK 2013:0267).

2.2.3.1 Compressive strength for bridges built before 1986 and at least 10 years old

For bridges built before 1986

The characteristic compressive strength f_{ck} from Table 1-5 must be adjusted according to equation 2.25:

$$f_{\mathrm{ckjust}} = 1{,}15 \times f_{\mathrm{ck}} - 2\ [\mathrm{MPa}] \tag{2.25}$$

For bridges built from 1986 and are at least 10 years old at the time of calculation.

The characteristic compressive strength f_{ck} from Table 1-5 must be adjusted according to Equation 2.26:

$$f_{\mathrm{ckjust}} = 1{,}15 \times f_{\mathrm{ck}}\ [\mathrm{MPa}] \tag{2.26}$$

2.2.3.2 Estimation of the characteristic compressive strength from results of cores

For bridges built according to the road traffic load regulations established during year 1994 and earlier and for bridges built according to railway traffic load regulations instituted during year 1999 and earlier

In order to evaluate the characteristic compressive strength from the results of the drilled cores, the sample must contain at least three drilled ø100 x 100 mm cylinders from the construction part that is to be tested. The value of K is obtained from Table 1-6 of the standard (TDOK 2013:0267). The required compressive strength values used in calculation for determining the characteristic strength values (f_{kk}) are as well presented in Table 1-6 in the standard.

There are different formulas that should be used for the calculation of the characteristic compressive strength. Equations (2.28) and (2.29) are used for both three and six samples, Equation (2.29) is used for three samples and Equation (2.30) is used for evaluation of six samples.

- Evaluation for three samples is done according to equations 2.26-2.28:

$$m \geq f_{\mathrm{kk}} + 4\ MPa \tag{2.27}$$

$$x \geq f_{\mathrm{kk}} - 5\ MPa \tag{2.28}$$

$$x \geq 0{,}8 f_{\mathrm{kk}} \tag{2.29}$$

- Evaluation for six samples is done according to Equations 2.29-2.31:

$$m \geq f_{\mathrm{kk}} \exp\left(1{,}4 s/m\right) \tag{2.30}$$

$$x \geq f_{\mathrm{kk}} - 5\ MPa \tag{2.31}$$

$$x \geq 0{,}8 f_{\mathrm{kk}} \qquad \text{where:} \tag{2.32}$$

m = mean value for all the strength values

s = standard deviation for all the strength values

x = strength value for individual sample

2.2.3.3 Bridges built according to the road traffic regulations established during year 1947 to 1960 or for bridges built according to railway traffic regulations set during year 1944 to 1960

The K- value and f_{ck} value is chosen according to the table in the standard but with additional rules:

- Concrete with K-values not exceeding K35 should be increased with three strength classes

- Concrete with K-values not exceeding K40 should be increased with two strength classes

- Concrete with K-values exceeding K40 is corresponded to the obtained strength class

2.2.3.4 Bridges built according to traffic regulations established during year 2002 and later

The compressive strength is calculated according to EN 1992-1-1 where the values of k_t and αcc are equal to one. If higher class than C60/75 is attained for compressive strength, the f_{ck} value is still obtained in each case.

The guidelines for evaluation of compressive strength for cores are referred to EN 13791.

According to 'Krav Brounderhåll: TDOK 2013:0415' the compressive strength of the hardened concrete must satisfy the requirements according to EN 13791. Additionally, the tensile strength must be at least 7% of the obtained compressive strength of concrete, yet not less than 6% of the nominal compressive strength.

2.2.5 The GCSC method

The GCSC method applies combination of probabilistic and empirical models to estimate the compressive strength of concrete in existing structures. The method was developed by Sangiorgio in 2018. This method is quick, cheap, adequately accurate and can be used with limited number of cores i.e., less than three. The method has been validated on various projects and showed consistency with the results of the actual strength of the concrete. The method offers an alternative in-situ compressive strength estimation tool for professional practice (Sangiorgio 2018).

For the estimation of the characteristic compressive strength, two graphs are used. One of the graphs represents the probabilistic distribution of characteristic in-situ strength of the concrete at the time of core drilling. The other graph depicts the development of the concrete strength from the time of construction. The development of the strength of the compressive strength is modelled depending on the age of concrete, exposure to environment and the age hardening effects. The number of cores required is determined by the purpose of the investigation and the size of the concrete to be tested. A test region, on the other hand, needs at least three cores to achieve acceptable accuracy. Cores should have a minimum diameter of 100 mm, and the length to diameter ratio are one or two. The test results must be checked statistically before being used for estimation purposes (Sangiorgio 2018).

2.2.5.1 Assumptions and graphical application of the method

The GCSC approach is based on the assumption that any result of core drilling compressive strength expressed in equivalent strength of standard cylinder is predicted to lie in the specified probabilistic limits. The range between the lower and upper characteristic values of $f_{\mathrm{c,is,cyl}}$ are: $f_{\mathrm{ck,inf,is,cyl}}$(5% fractile) and $f_{\mathrm{ck,sup,is,cyl}}$ (95% fractile). Further on, the probability that lies in the interval $f_{\mathrm{cm,is,cyl}} \pm \sigma$ is approximately 70% of any CSC strength. The *Figure 2.56* below depicts graphically the probabilistic distribution assumptions for CSC (Sangiorgio 2018).

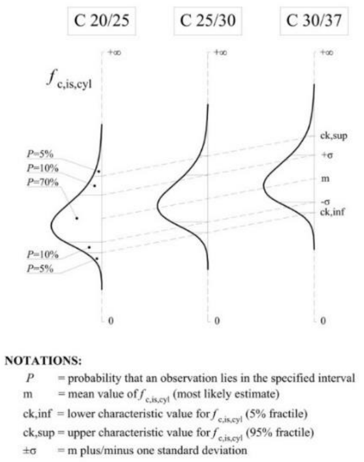


Figure 2.56: Probabilistic distribution of $f_{c,is,cyl}$ (Sangiorgio, 2018)

As mentioned earlier, the GCSC method employs two graphs for the assessment of the compressive strength classes. The first graph is used to estimate the in-situ compressive strength class or classes from five regression lines which represent the following probability of occurrence of the strength class

1) 90% probability of occurrence- The upper and lower boundary lines of $f_{\mathrm{c,is,cyl}}$,that is $f_{\mathrm{ck,inf,is,cyl}}$ and $f_{\mathrm{ck,sup,is,cyl}}$
2) 70% probability of the CSC: -The probability lines that contain the CSC which lies in the range $f_{\mathrm{cm,is,cyl}} \pm \sigma$
3) The most probable CSC- This is the line representing the mean of the CSC

The concrete strength at the construction time is determined according to the corresponding age hardening effects (AHE) of the concrete from the 28th day of the casting of the concrete until the time of core drilling. Thus, the concrete strength class at the time of the construction is estimated from the equation 2.33 by considering the time backwards to the construction (Sangiorgio 2018).

$$f_{\mathrm{c,is,cyl}}(t_0) = f_{\mathrm{c,is,cyl}}/\beta_{\mathrm{cc}}(t^-) \qquad \text{where:} \tag{2.33}$$

$$\beta_{\mathrm{cc}}(t^-) = \exp\{s[1-(28/t^-)^{1/2}]\} \tag{2.34}$$

The assessment procedure for classification of the concrete into concrete strength class or classes is started by the determination of the equivalent standard core strength $(f_{\mathrm{c,is,cyl}})$ according to EN 12504-1. Subsequently, the determination of the CSC from the graph in *Figure 2.57* follows by drawing horizontal stroke from $f_{\mathrm{c,is,cyl}}$ throughout the whole range of strength classes. The line intercepts the five boundary lines. A line drawn vertically to intercept the abscissa from the intersection with the line of the upper bound of the characteristic strength ($f_{\mathrm{ck,sup,is,cyl}}$) indicates the lower strength bound of the CSC (1), C_{inf} on the abscissa.

Similarly, a line drawn vertically towards the abscissa from the interception with the lines with 70% probability of characteristic strength and drawing two lines indicate two extreme bounds of the CSC (2a and 2b) with 70% of probability of occurrence interval ($f_{\mathrm{cm,is,cyl}} \pm\sigma$), C_{m}. A line drawn from the interception with the line of most probable strength and vertically to the abscissa indicates the most probable CSC (3), C_{m}. Finally, the horizontal stroke that intersects with the line of the lower bound of characteristic strength and extended vertically towards the abscissa indicates the higher bound of the CSC (4), ($f_{\mathrm{ck,sup,is,cyl}}$),$C_{\mathrm{sup}}$ (Sangiorgio 2018).

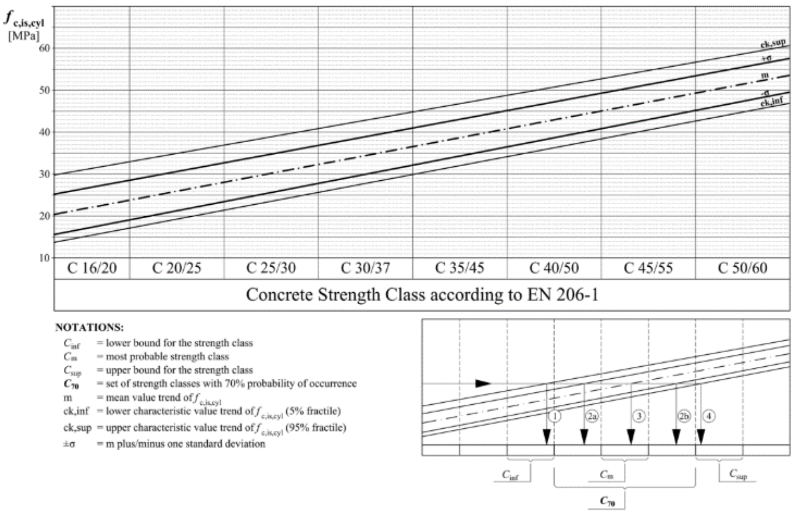


Figure 2.57: Compressive strength classes for different probabilistic expectations (Sangiorgio 2018)

2.2.5.2 Assessment of CSC according to GCSC

The design concrete strength estimates of C_{d} and $C_{\mathrm{o,d}}$ are approximated according the specified rules. Those values may also be provided as design inputs. The number of cores has an effect on the precision of the estimation. If the evaluation yields values that are too low or not safe, the number of cores need to be increased (Sangiorgio 2018).

The initial inputs of core cylinder strength are determined depending on the number of cores as follows:

- for one or two drilling in a test region, determine the $f_{\mathrm{c,is,cyl}}$ and $f_{\mathrm{c,is,cyl}}(t_o)$ for each test result
- for three or more drilling in a test region, determine the $f_{\mathrm{c,m,,is,cyl}}$ and $f_{\mathrm{c,m,,is,cyl}}(t_o)$ for each test result (Sangiorgio, 2018).

The estimation of design strength according to GCSC depends on the number of cores. The equations 2.33-2.37 below are used to determine the CSC:

1. One drilling core:
   - The ranges of strength classes C and C_{o} is given by:

   $C_{\mathrm{inf}} \leq C \leq C_{\mathrm{sup}}$
   $C_{0,\mathrm{inf}} \leq C_0 \leq C_{0,\mathrm{sup}}$
   The reasonable design range for C and C_{o} shall be:

$$C_{\mathrm{d}} = \min\{\mathbf{C}_{70}\} \tag{2.34}$$

$$C_{0,d} = \begin{cases} \min\{\mathbf{C}_{0.70}\} & \text{for significant AHE} \\ \mathbf{C}_{0,\mathrm{m}} & \text{for moderate AHE} \\ \mathbf{C}_{\mathrm{d}} & \text{for negligible AHE} \end{cases} \tag{2.35}$$

2. Two drilling cores:

   - The most probable ranges of classes of C and C_{o} is reduced slightly than with one drilling core and are given as follows:

   $\max\{C_{\mathrm{inf,i}}\} \leq C \leq \min\{C_{\mathrm{sup,i}}\}$
   $\max\{C_{0,\mathrm{inf,i}}\} \leq C_0 \leq \min\{C_{0,\mathrm{sup,i}}\}$
   Where i is index of the elements in a set

   Approximate values of design range applicable in wide range of cases are given in equations below.

$$C_{\mathrm{d}} = \max\{C_{\mathrm{inf,i}}\} \tag{2.36}$$

$$C_{0,\mathrm{d}} = \begin{cases} \max\{C_{0,\mathrm{inf,i}}\} & \text{for significant AHE} \\ \max\{C_{0,\mathrm{m,i}}\} & \text{for moderate AHE} \\ C_{\mathrm{d}} & \text{for negligible AHE} \end{cases} \tag{2.37}$$

3. 3 or more core drillings:

C is unique and is evaluated from $f_{\text{cm,is,cyl}}$
C_0 is expected to fall in the inequality (5). However, it is estimated from $f_{\text{cm,is,cyl}}(t_0)$
$C_{0,d}$ is defined by the following expressions

$$C_{0,\text{d}} = \begin{cases} \min\{C_{0.70}\} & \text{for significant AHE} \\ C_{0,\text{m}} & \text{for moderate AHE} \end{cases} \tag{2.38}$$

2.2.5.3 Netherland study based on EN1990

The method of approach B in EN 13791:2007 leads to unsafe results of characteristic strength. A paper presented by Vervuurt et al. introduced a method based on EN 1990 and is validated on 200 structures in highways in Netherland.

EN 1990 is a general guideline for design that considers the statistical uncertainty, the mean and standard deviation. When sample of cores are normally collected, the areas that are stressed which should have been aimed for, are avoided in order not to damage the structure. The area covered could also be so small that it can only indicate the local variation and not the overall variation of the whole structure. The standard deviation could be low because the investigated area may not cover all of the concrete.

Hence, other way of introducing minimum standard deviation by using probabilistic prediction from large set of data, can result in safer estimation of compressive strength of in situ concrete. The *Figure 2.58* shows that the compressive strength estimated by EN 13791:2007 is always higher for 200 cases of structures in a Netherlands study. It was concluded that the high results are because of the methodology of assessment (Vervuurt et al. 2013).

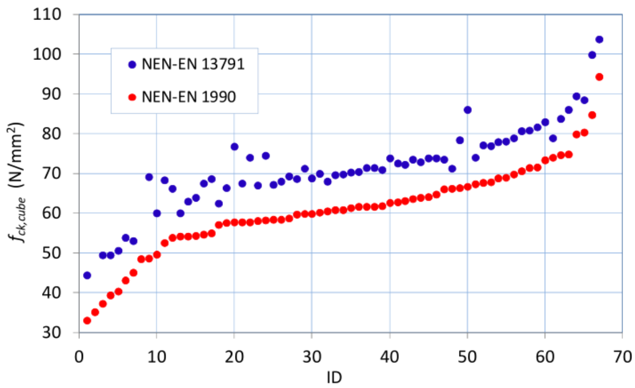


Figure 2.58: The comparison of estimates of the characteristic compressive strength according to EN 13791 and EN 1990

The characteristics in situ compressive strength is computed as the minimum of two methods

Method A: $f_{ck,cube} = exp\{f_{cm,cube}(Y)\}.exp\left\{-t_{n-1}(p=0{,}05)\cdot s(Y)\cdot\sqrt{1+\frac{1}{n}}\right\}$

(2.39)

Method B: $$f_{\mathrm{ck,cube}} = \exp\{f_{\mathrm{cm,cube}}(Y)\}.\exp\left\{-1.64 \cdot s_{\min}(Y) \cdot \sqrt{1+\frac{1}{n}}\right\} \tag{2.40}$$

Where:

$f_{\mathrm{ck,cube}}$	In situ cube characteristic compressive strength
$f_{\mathrm{cm,cube}}(Y)$	Mean of the logarithm of in situ core test results
n	number of cores
$s(Y)$	Standard deviation of the logarithm of in situ core test results
$t_{\mathrm{n-1}}$	The value if t distribution for degree of freedom n-1

The logarithmic value of the $s_{\min}$ can be calculated from: -

$$s_{\min}(Y) = \sqrt{\ln\left(1+\left(\frac{s_{\min}}{f_{\mathrm{cm,cube}}}\right)^2\right)} \tag{2.41}$$

$s_{\min}$ represents minimum standard deviation recommended from the results of experiments.

The minimum standard deviation is estimated from results suggested from experiments performed on 200 structures in Dutch highways. The *Figure 2.59* or suggested values according to *Table 2.5* types of structures can be used. A probability of exceedance of 50% is recommended for safe estimate.

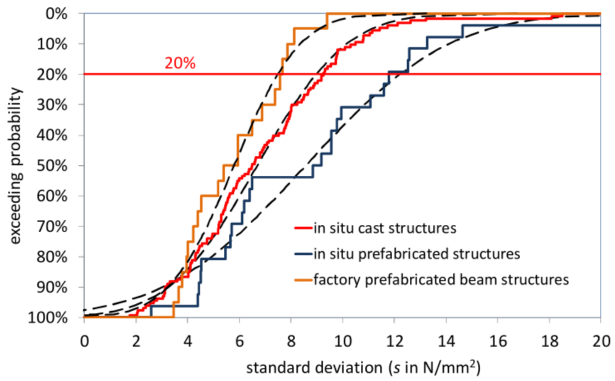


Figure 1.59: The probabilistic distribution of standard deviation of 200 structures in highways in Netherland

Table 2.5 Values of $s_{\min}$ for different types of structures

Type of structure	$s_{\min}$
In situ structures	10
in situ prefabricated structural elements	12
in a factory prefabricated structural elements	8

2.2.5.4 DIN EN 13791:2017

The approach B of EN 13791:2007 leads to overestimation of results. A modified version of the standard is in use in Germany which results in safe values for small number of cores. This new procedure is based on *DIN EN 13791: 2008-05*, which modified approaches A and B is now part of *Germany standard DIN EN 13791 / A20: 2017-02*. This standard has provision for estimation for number of cores as small as 3 and the results are stated to be safe for use of structural calculations. The method is developed from a study which showed that limiting the coefficient of variation to a minimum of 0.08 (v_x =0.08) can prevent overestimation of the results. Whereas, the limitations of the scatter to a maximum coefficient of variation, of 0.2 (v_x =0.2), limits the under estimation even though almost 90% of samples are excluded (Weber 2020).

As a result, the estimates of the characteristic strength for v_x >0.2 will be on the safe side and the use of approach B leads to low spread of results for $v_x \leq 0.2$. For number of cores of three, the results can be reduced or exceeded by to 65%-190%. Similarly, for number of cores of eight, the results can be over or under estimated to 80-170%. The maximum overestimation observed for samples with a coefficient of variation of v_x<0.2 is normally less than 10 N / mm^2. On the other hand, overestimations of approximately 20 N / mm^2 may occur in the case of large scatter (Weber 2020).

Finally, the limitation over coefficient of variation results in more effectiveness compared to minimum standard deviation in reducing the scatter. The standard deviation limitation is effective for small samples and small scatter. The minimum value of the coefficient of variation of 0.09 (v_x =0.09) was chosen so that the results are safe and economical for practical application (Weber M 2020).

<u>Modified approach A</u>

The characteristic in situ compressive strength is estimated using equation 2.42.

$$f_{\mathrm{ck,is}} = f_{\mathrm{m(n),is}} \cdot (1 - k_{\mathrm{n}} \cdot v_x) \tag{2.42}$$

The modified approach A is to be applied for a minimum of nine cores or for a minimum of three cores when the coefficient of variation is greater than 0.2 (v_x > 0.20).

<u>Modified approach B</u>

The modified approach B is to be used for cases with three to eight cores and the coefficient of variation of the sample has a maximum value of 0.2 ($v_x \leq 0.20$).

The coefficient of variation of the sample is limited to minimum value of 0.09 and the factor k_{n} is the same factor used in EN 1990. The characteristic in situ compressive strength is estimated from the lower of equations 2.43 and 2.44.

$$f_{\mathrm{ck,is}} = f_{\mathrm{m(n),is}} \cdot k_3 \tag{2.43}$$

$$f_{\mathrm{ck,is}} = f_{\mathrm{is,lowest}} + 4 \ \mathrm{Mpa} \tag{2.44}$$

Table 1.6: The values of k_3 for ranges of number of cores

n	K_3
3	0.7
4-5	0.75
6-8	0.8

$$k_3 = 1 - k_{\mathrm{n}} \cdot v_x \tag{2.45}$$

The factor k_3 given in *Table 2.6*, is derived from *EN 1990: 2010-12* for v_x, min =0.09 according to Equation 2.45

3 Methodology

3.1 Case study of Skuru bridge

The study of the thesis is based on a case study of the Skuru bridge which is located in the Nacka community in Stockholm. The southern bridge is constructed in 1914 while the Northern bridge is constructed in 1957 (Sangiorgio 2020).

According to the investigation report of COWI, some rehabilitation works were executed in the southern bridge at the time the Northern bridge was constructed. The study concerns the Southern bridge which is under process to renovate and make it suitable for the use of the current traffic (Sangiorgio 2020).

Figure 3.1: Illustration of Skuru bridge, (Wikipedia)

3.1.1 Previous investigations

The study of the thesis project is based on the previous investigations carried out by COWI. The previous detailed investigation report by COWI dated 2020-12-18 is adopted as the source of the data pertaining previous investigations. The report also describes the results of the compression tests of the samples of drilling cores, which are performed according to EN 12390-3 and EN 12390-6.

The concrete data from the time of construction that is obtained from the report is presented in *Table 3.1*

Table 3.1: Concrete data

Structural element	Concrete mix	Concrete strength class	W/c	Age
Column 1914	1:3:3	σ_{B28} 160	0.7	106
Arch	1:3:3	σ_{B28} 160	0.7	106

3.2 Planning and undertaking of tests

First an in-depth literature study and assessment of the different test methods with respect to their advantages and limitations was performed. Afterwards, the Pull-out method was chosen because it assesses more depth of concrete and does not get affected by many factors. Further, the Schmidt hammer was also opted because it is a simple, widely used and quick method that give good measurements of the mechanical properties of the concrete.

Unfortunately, due to shortage of supplier of the Pull-out instruments, the test method could not be applied. The Schmidt hammer and UPV were later applied to the Skuru bridge in order to perform non-destructive tests to assess the quality of the concrete .

3.2.1 Planning and testing of destructive tests

According to Filippo Sangiorgio who was in charge of preparing the investigation, several influencing factors are assessed in the selection of the core locations. The core locations were planned in such a way that any cracks are avoided to prevent inducing further cracking that can endanger the safety of the structure. The other consideration was the access to the locations since the movement and the operation of the test apparatus can incur more labor and cost if the locations have difficulty of access. The planning of the test locations was also determined by consideration of the variability of the concrete to be investigated. The locations of the drilled coring tests and non-destructive tests are illustrated in Appendix 8.9.

3.2.2 Planning and testing of non-destructive tests

Before conducting the tests, a standard form was prepared in order to record every necessary data such as: environmental conditions (temperature and relative humidity), time of test, test locations, number of tests, concrete cover and type of test.

The non-destructive tests were planned to investigate the quality of concrete and estimate the characteristic concrete strength using correlation. However, due to the limitation of the indirect tests, the tests were only performed on six locations and those tests made on the same locations as the cores were four.

The measurements were performed with the supervision of the supplier KmK Instrument. The measurements were performed on four columns and one arch according to the drawing in Appendix 8.9; namely column 4 North, column 5 North, column 4 South, column 5 South, arch one and arch two.

The tests were supposed to include columns at the position of 26 in the drawing, but unfortunately these columns were not accessible. Further on, there was limitation of time and therefore the tests were only performed on few locations.

Before the measurements with the ultrasonic pulse velocity and Schmidt hammer were conducted, a GPR (ground penetrating radar) was used for locating the reinforcement in the concrete. The GPR(GP8000) scan is employed to ensure that the drilling cores are safely away from the reinforcement. The scan was made only on column 4 north. The GPR was only a demonstration for how to precisely locate the drill core tests.

The line scan and area scan methods of the GPR scanning are applied on the columns mentioned. The line scan method gives the information of the location of the reinforcement in 2D and augmented reality. On the other hand, the area scan shown in *Figure 3.5* provides the location of reinforcement in 3D and augmented reality. The results of the tests are presented in *Figures 4.1-4.6*

Afterwards, the measurement with Schmidt Hammer tests were commenced as shown in *Figure 3.2*. A set of nine measurements were made for each test according to the European standard. The tests were accomplished on the designated columns and arch locations according to Appendix 8.9: Column 4 North(4N), Column 4 South(4S), Column 27 North(27N), Column 27 South(27S), Arch one and Arch two.

The measurements of the Schmidt hammer require the investigated concrete surface to be smooth and that the right concrete should be identified. The surface of the arch consists of a finishing concrete that is different from the structural concrete of the bridge. Therefore, suitable locations were selected by avoiding the unrepresentative finishing concrete. The selected areas for the arch consisted of a side of the arch and an area on the surface where the finishing concrete was peeled off and the appropriate concrete was exposed.

Further on, the UPV was applied on the same construction members excluding the 4 South and 5 South. There is no specified requirement about the number of tests in the EN standards. Therefore, sets of four measurements were carried out for each test location, except in the arch one, where three measurements were taken. A test being performed with UPV is shown in *Figure 3.4*.

Finally, all results were stored in the devices and the Kmk Instrument provided us with all the outcomes from the readings.

Figure 3.2: Schmidt hammer tests

Figure 3.3: Schmidt Hammer device

Figure 3.4: UPV-test: direct transmission

Figure 3.5: Area scan

3.2.3 Statistical analysis procedures

According to EN 12504-2, the reading of rebound number of a set of measurements is the median. The set of readings should fulfill that not more than one reading should differ from the median by 30%. The validity of the data of each set of readings is checked to comply with the requirements according to EN 12504-2. The Appendix 8.4.1 presents the results of the validation checks.

The Grubb test that is used to remove outliers is as per the procedures of the EN 13791:2019 for the core results and the UPV readings. The Grubb test is based on Gaussian distribution and the guidance to application of EN 13791, CEN/TR 17086 for core drilling test, rebound hammer test and UPV results can be assumed to form normal distribution or lognormal distribution. Hence, this assumption is utilized for those data to treat the outliers. The check of the initial data set for the core results and UPV readings are presented in Appendix 8.4.2.

3.3 Determination of test regions

Guidance about the formation of test regions is not included in EN 13791:2007. According to the EN 13791:2019, the selection of test regions depends on the extent of information about the construction of the structure in the construction documents. Depending on the previous information from the construction such as mix design, concrete type and type of elements, different test regions can be identified. In case there is lack of prior information, engineering judgment or indirect test results can indicate regions of the same concrete quality.

In the case of the Skuru bridge the original construction drawings provide the information about the concrete strength at the time of construction. According to the drawings the type of concrete used had the same strength for the columns and the arch. Thus, it can be determined that the columns and the arch form a single test region.

3.4 Simulations of samples with varying number of cores

The results of the core drilling tests can be considered as random variables of a normal or lognormal distribution. Hence, the simulations of different number of cores are based on the assumption that there is random probability of the results. The random combinations of samples consisting of different number of cores are utilized for assessing the variations or spread of the estimations using the various methods.

A Matlab code is used for generating simulated samples of combination of three to eight cores. The Matlab codes used are attached in Appendix 8.5. The characteristic in-situ compressive strength of the simulated samples of the combinations are computed using Excel sheet and the spread of the results are plotted.

The estimation of the characteristic in-situ strength using small number of cores is allowed in EN 13791:2007, TDOK, the graphical method and the DIN 13791:2018 modified method from Germany. Hence, the scatter of the estimation from the samples generated by combining number of cores from three to eight is estimated according to those standards.

3.5 Estimation of in-situ concrete strength according to TDOK

As mentioned above, the bridge is constructed in 1914. In TDOK, the clause that concerns the construction period is applied. For calculating the characteristic compressive strength of concrete, equations 26-28 are used. The calculations consisted of first obtaining the value of F_{kk} (using Eq. 2.29). Thereafter, this value is checked against each core value using equations 2.30 and 2.31. If the value of F_{kk} fulfills the conditions, the K-value is determined according to T*able 1-6* from the standard.

3.6 Estimation of in-situ concrete strength according to EN 13791:2007

There are two methods for estimating the characteristic compressive strength of concrete according to the standard. The first method is only based on the drilled cores, but there are two different approaches depending on the number of available cores. In this study there are nine accessible cores and according to clause 7.3, approach B should be used, hence calculations according to approach B have been completed.

Further the variations of the estimation of the characteristic strength is evaluated by simulating of combinations of three to eight core samples from a total population of nine cores.

The other method for calculating the characteristic compressive strength of concrete is through calibration or correlation depending on the number of the available pair results. In this case there are four measurements from the UPV and rebound hammer results that correspond to the four cores taken from the same locations, meaning that there are four available pair of results. The pairs of results comprise of core and UPV or Rebound hammer results from column 4 North, 4 South, arch one and arch two. Clause 8.2 in the standard states that the correlation method should be used when there are 18 pairs of results, meaning that this approach is not applicable. However, to demonstrate the application, the correlation is established of those available pairs of data. The characteristic in-situ strength according to correlation is estimated but the accuracy is unreliable because of inadequate pairs of data.

3.7 Estimation of in-situ concrete strength according to EN 13791:2019

The number of drilled core tests performed in the columns and the arch of the entire bridge is nine. The number of indirect tests that are located where there is coring is four. Hence, the clause 8.1 which requires a minimum of eight cores is satisfied. The values of the results from the compression tests are changed to equivalent core strength to length to depth ratio of 2:1 according to the procedure in the standard. Those converted values are assessed for outliers by applying the Grubb test. The procedures of calculations according to clause 8.1 is followed to estimate the characteristics in-situ strength.

The clause 8.2 provides guidance on correlation of indirect and coring test results with minimum number of eight cores. The correlation of indirect and direct tests is helpful particularly when there are many test regions. The pair of data available for correlation in the case of this study is four. Hence, the requirement set by the standard is not met. However, in this study the correlation is considered to demonstrate its application and compare the results.

3.8 Estimation of in situ concrete strength according to the GCSC method

The estimation of characteristic strength proceeds after the test region is identified. The use of all core test results for the estimation purpose increases the precision of the estimated concrete strength class.

Estimation of the characteristic in-situ strength of the test regions depends on the number of cores considered. For one and two cores, the equivalent core strength is used to determine the concrete strength class according to the rules of the method. In the case of three and more cores, the mean of the core strength results is employed to determine the characteristic strength class.

The graphical method is applicable for any number of cores and the results become more accurate with the increasing number of cores. The variation of the use of single cores is analyzed by using the method for estimation of the strength based only on each core result separately. The variation of using two cores is assessed by the combination of a pair core in the sample of nine cores. The variation of using three to eight cores is also assessed by combining them in sets of three to eight cores. It is assumed that the estimation of the strength class by using nine cores gives comparatively more accurate results and this result is applied to verify the degree of variability with use of more cores in the application of GCSC method.

3.9 Estimation according to DIN 13791:2017 and Netherland study

The characteristic in-situ compressive strength is calculated using the equations 2.39-2.41 for the Netherlands method and equations 2.42-2.45 for the DIN 13791:2017. When the number of cores is nine, DIN EN 13791:2019 procedure becomes the same as EN 13791:2019.

Further, the use of three to eight cores is assessed by simulation of combinations from the nine cores as mentioned in section 3.4. The estimation of characteristic in-situ compressive strength for the combinations of the samples is done according to DIN EN 13791:2017. For the Netherland method, the characteristic in-situ strength of the combinations of the samples is estimated for the particular number of cores using the corresponding t-value from standard t-distribution table in equation 2.39. The minimum standard deviation that is used in the Method B is adopted from Table 2.5.

4 Results

4.1 Results of non-destructive tests

4.1.1 Ground penetration radar (GPR) scan test result

The Ground penetration scan is performed only on Column 5N. The purpose of the test was to determine the cover of the reinforcement location. The results of the line and area scans are presented in the following sections.

4.1.1.1 GPR line scan results

The line scan results provide the location of the object at depth, longitudinally or transversally. The sound waves are reflected from the surface of reinforcement and makes the raw data or the migrated view as in *Figure 4.1-4.2*.

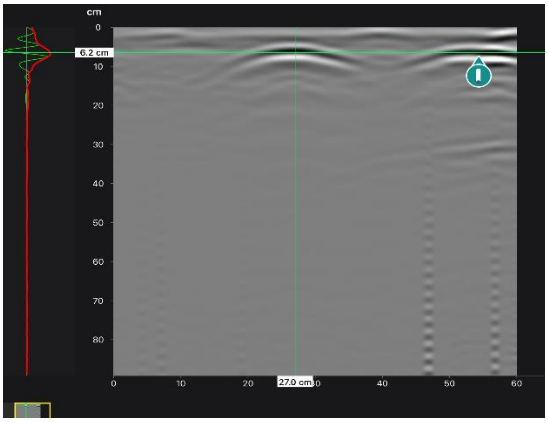


Figure 4.1: Raw data or hyperbola view of GPR scan

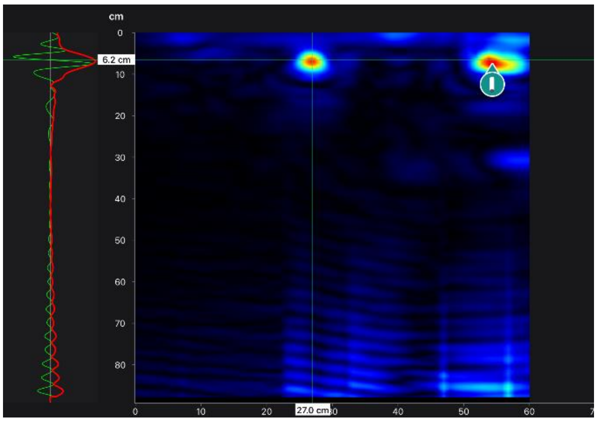


Figure 4.2: GPR- migrated view

4.1.1.2 GPR area scan results

The area scan results capture the location of the object in 2D and 3D.

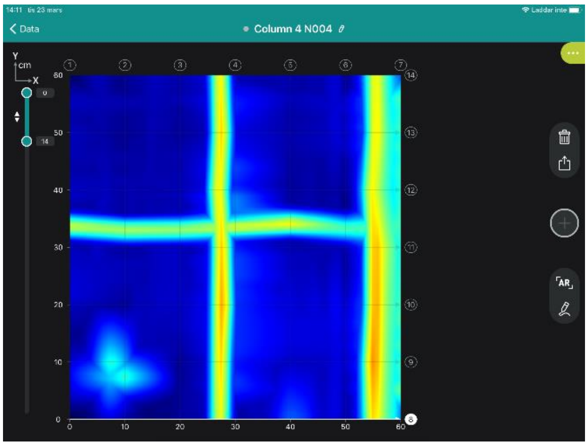


Figure 4.3: GPR 2D-view

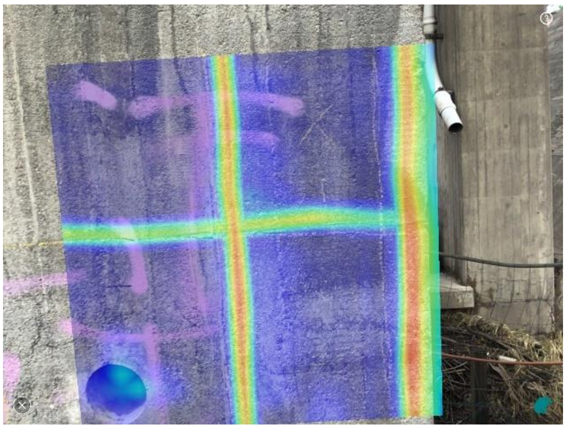


Figure 4.4: 2D augmented reality

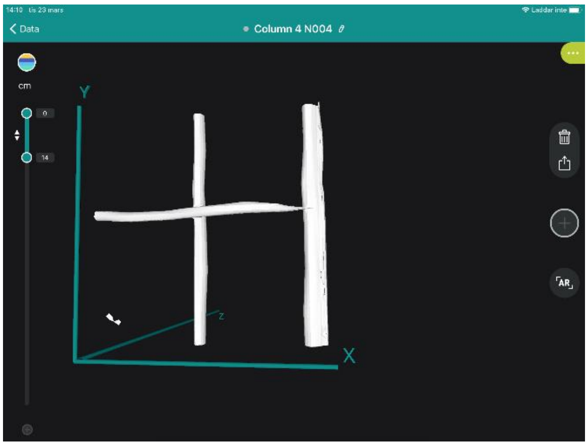


Figure 4.5: 3D-view

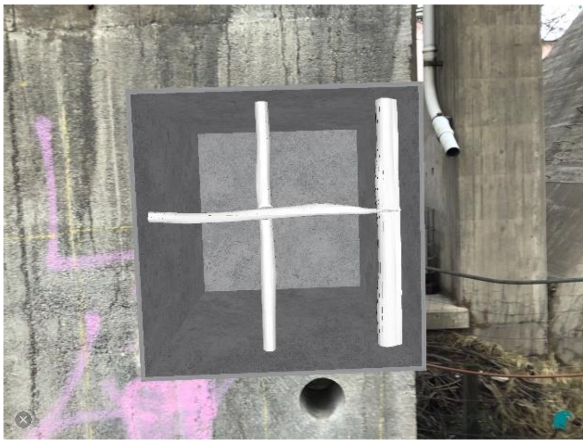


Figure 4.6: 3D augmented reality

4.1.2 Rebound hammer test results

The median of nine readings of the Schmidt hammer test is considered as the test result according to EN 12504-2. The following results presented in *Figure 4.7* are the median of each set of reading. The results are checked for the requirement in the standard that no reading lies further than 30% of the median as presented in Appendix 8.4.1. The final results of Schmidt hammer test were consistent and that shows the homogeneity of the concrete. The individual results of the test are presented in Appendix 8.3.

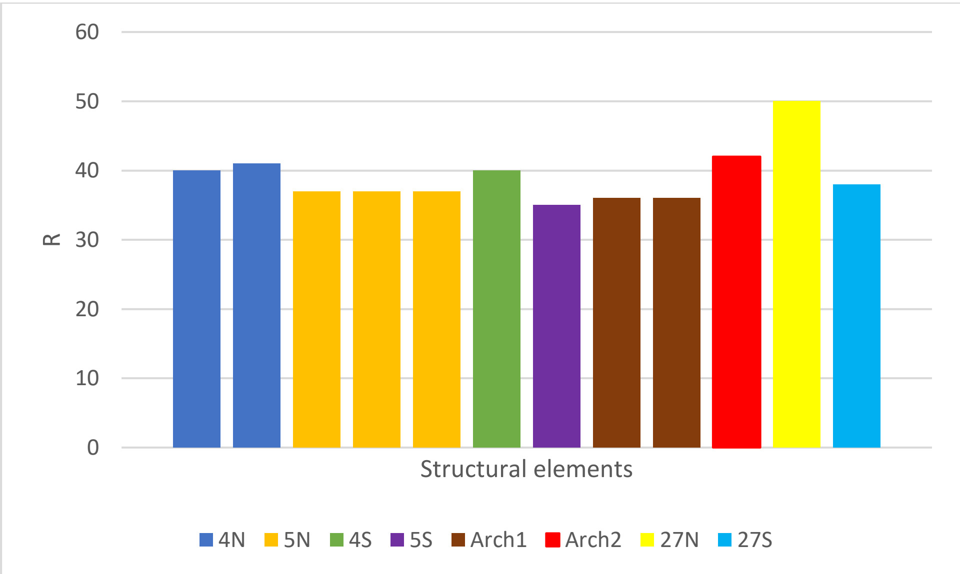


Figure 4.7: Rebound hammer test results

4.1.3 Ultrasonic pulse velocity test results

The initial data of the ultrasonic pulse velocity results are analyzed for the outliers by applying the Grubb test according to EN 13791:2019 recommendations. All results comply the set limits by the Grubb test. The validation check is attached in Appendix 8.4.2.

Figure 4.8 presents the results of the UPV measurements. The following results are somehow stable despite the results from Arch 1. The UPV-results for each element are separately presented in Appendix 8.2. The UPV results showed good consistency except the lower result that was obtained at one test location in the arch.

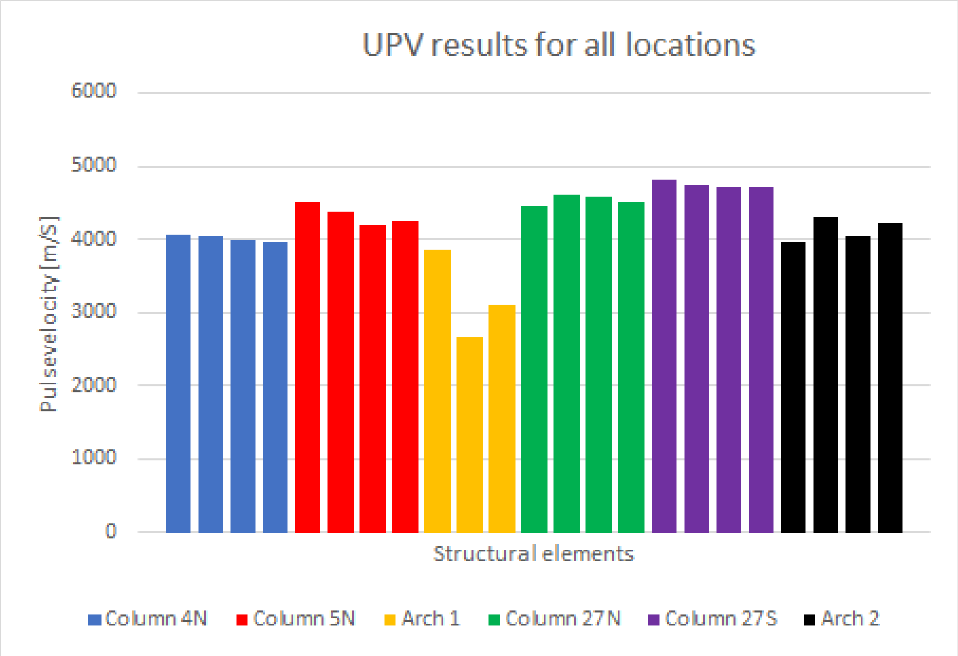


Figure 4.8: UPV results for all locations

4.2 Results of destructive tests from previous investigations

The results in equivalent cube and cylinder compressive strength are presented in *Table 4.1*.

Table 4.1 Results of compressive in-situ strength of drilling cores

Test location	**4N**	**4N**	**4N**	**5N**	**Arch 1**	**Arch2**	**26N**	**26N**	**26N**
Serial number	**1**	**2**	**3**	**1**	**1**	**1**	**1**	**2**	**3**
$f_{c,1:1\ core}$	28.1	47.1	37.4	40.2	45.8	33.4	33.7	43.4	38.3
$f_{c,is}$	23.0	38.6	30.7	33.0	37.6	27.4	27.6	35.6	31.4

4.3 Results of estimation of concrete strength

When all the cores are utilized in estimation of the characteristic in-situ compressive strength, different results are obtained. The estimated characteristic in-situ compressive strength class according to TDOK and EN 13791:2007 is C30/37. For EN 13791:2019, the in-situ characteristic concrete compressive strength class obtained is C20/25 if all nine cores are used. Moreover, the result for the graphical method is C25/30. The Netherland study and the DIN 13791:2017 resulted in concrete strength class of C20/25. The results are presented in *Table 4.2* and *Figure 4.9.*

Table 4.2 Results of characteristic in-situ CSC according to different methods of interpretation

Method	Reference clause	CSC
EN13791/2019	8,1	20
EN13791/2007	7.3.3, Approach B	30
GCSC all cores	Sveral cores method	25
TDOK	1.3.2.1.2	30
Netherland study	(Modified EN 1990)	25
German standard	DIN 13791:2017, Method A	20

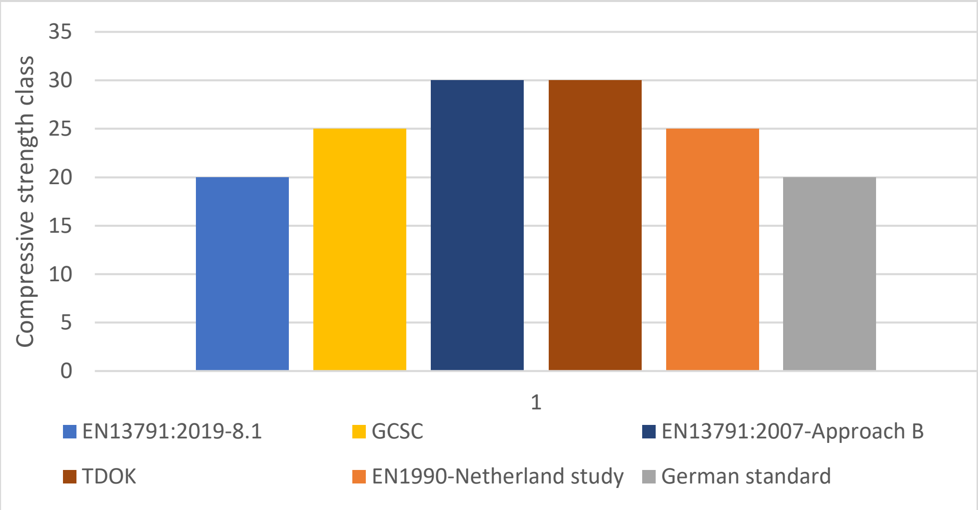


Figure 4.9 Estimate of characteristic in-situ CSC according to different interpretation methods

The result from the correlation of the UPV and the Rebound hammer according to EN 13791:2007 and EN 13791:2019 are given in Appendix 8.1.5-8.1.7. However, the results are unreliable since the number of cores required for correlation is not satisfied.

The graphical method can be used for any number of cores. The method is applied to each core and to combinations of two to eight cores to evaluate the scatter of the results. When all the cores are utilized in the estimation, the obtained result is C25/30. For the GCSC, when the number of cores is two the results are underestimated. In the case of one to three cores, low or high values of results could be obtained. When more than three cores are considered in the estimation, consistent results are obtained. The *Figure 4.10* depicts the results for different number of cores.

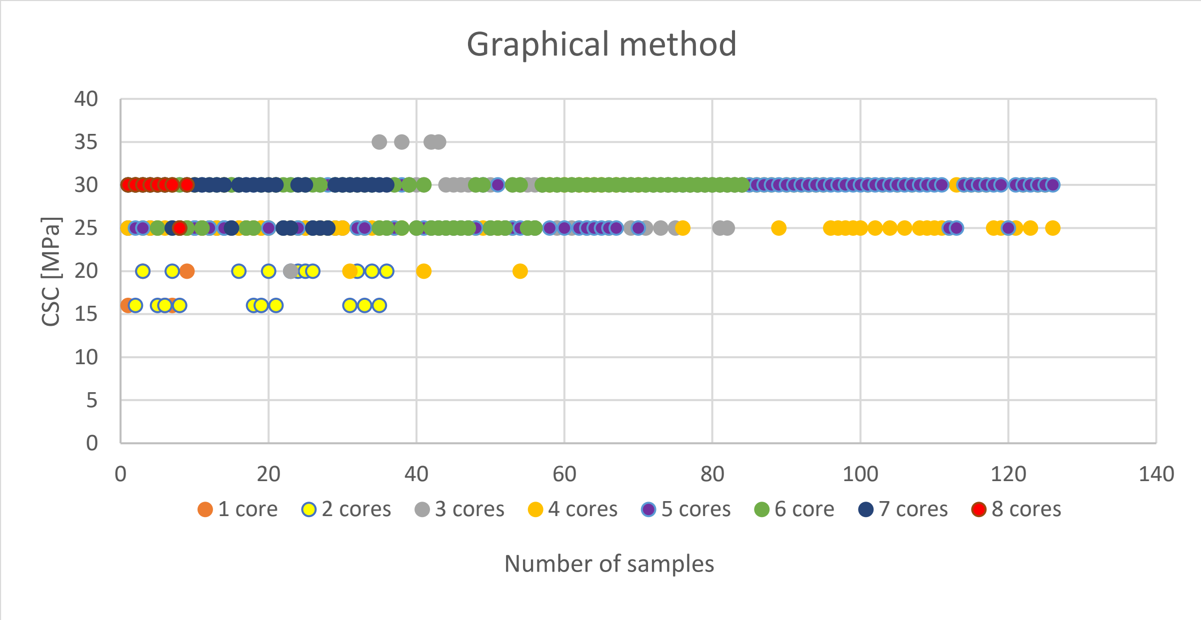


Figure 4.10: The results of the GCSC for combinations of different number of cores

The use of three cores for estimation of characteristic in-situ CSC is possible in the EN 13791:2007, DIN 13791:2017, GCSC and the Netherland study. Comparatively the Netherland study and DIN 13791:2017 give safer results but can sometimes be underestimated. However, higher results could be obtained using EN 13791:2007 and TDOK. The results are shown in *Figure 4.11*.

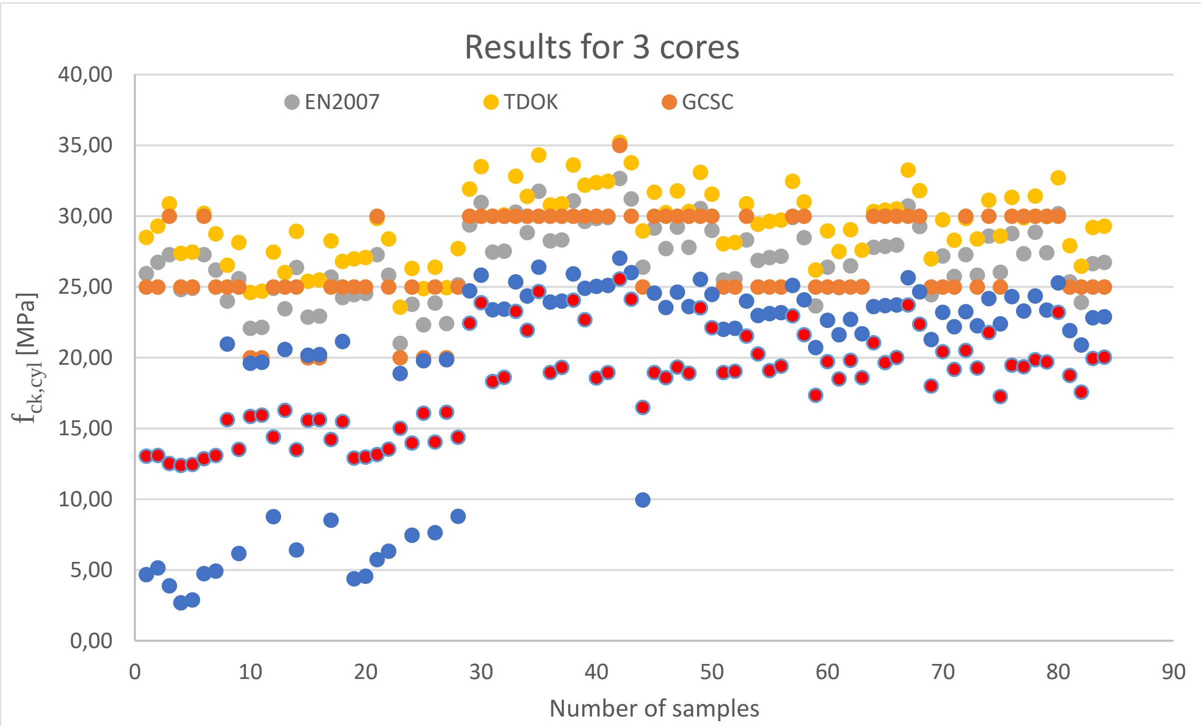


Figure 4.11: Results of combining three cores according to different standards.

The estimation of the CSC using EN 2007 and TDOK show large spread for three cores and the spread decreases to low values as the number of cores increase. Relatively, those methods give higher results. The scatter of the maximum estimate compared to the results of nine cores is 6,4 MPa for three cores, 4,3 MPa for four cores and 3,3 MPa for five cores. The results of the estimations of three to eight cores are presented below in *Figure 4.12* and *Figure 4.13*.

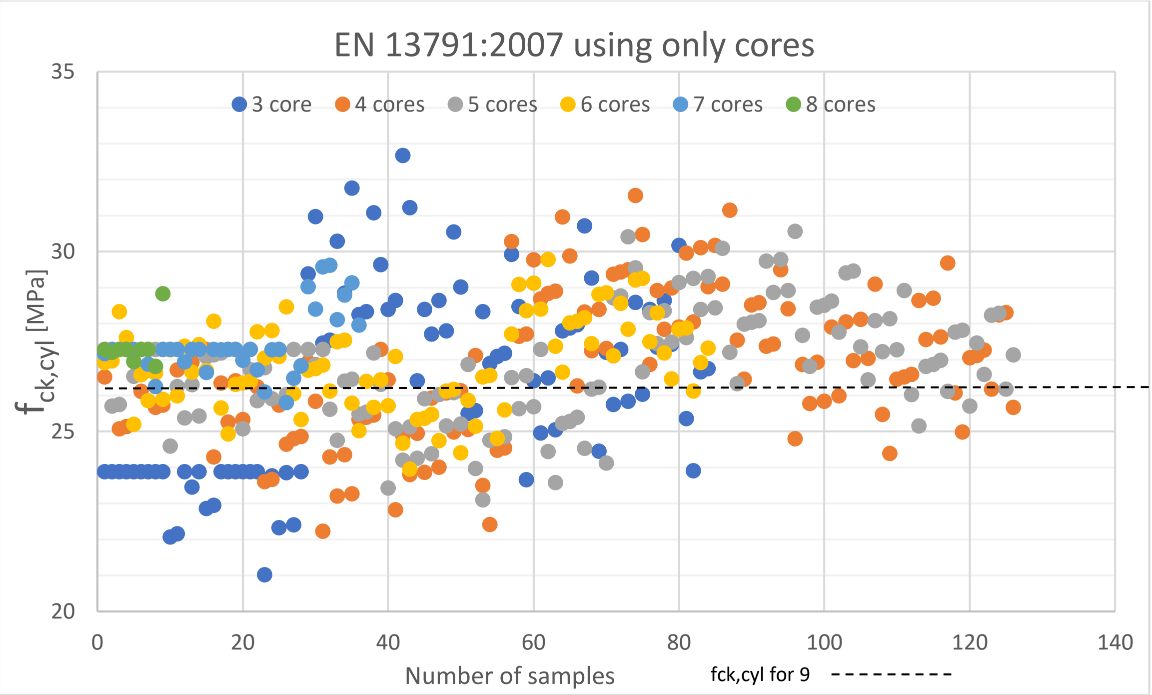


Figure 4.12: Results of three to eight cores according to EN 13791:2007.

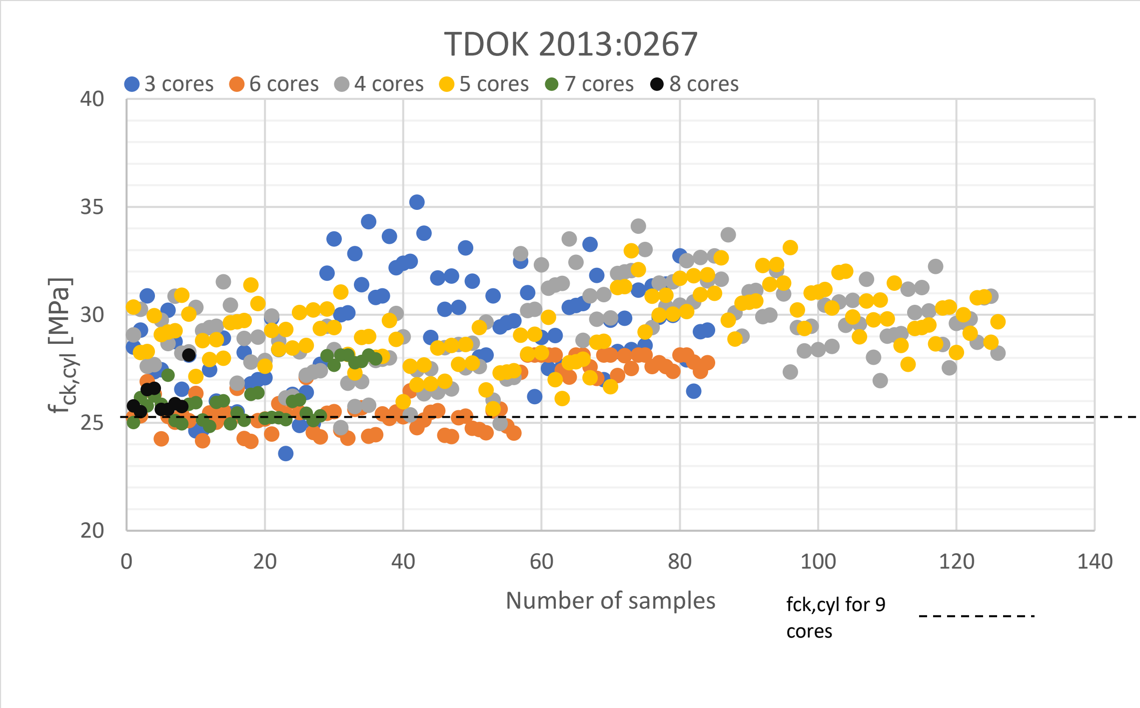


Figure 4.13: Results of three to eight cores according to TDOK 2013:0267.

As shown in *Figure 4.14*, the results from eight cores are close to the result of nine cores.

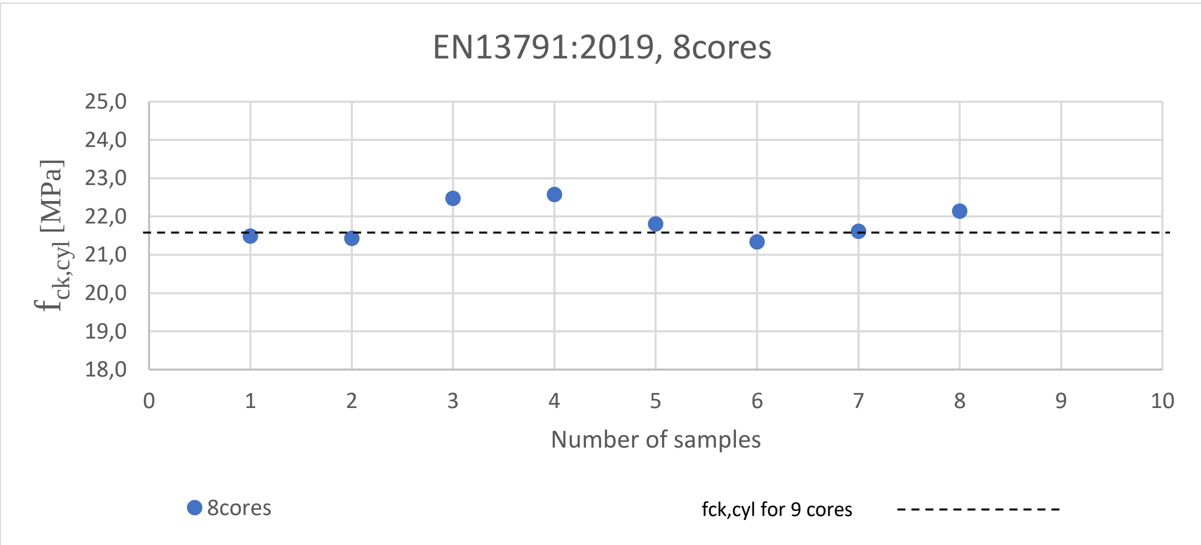


Figure 4.14: Results from eight cores using EN 13791:2019.

The scatter from the German standard DIN 13791:2017 is large when the number of cores is three to five resulting in low estimates. However, the scatter is low for number of cores exceeding five. Though there is large scatter, the estimation using three cores and four cores most of the time gives safe values. The maximum estimates of the results from the number of cores three to eight is 28 MPa. Further very low results are observed for three to five cores with values as low as 3 MPa. The results for combining three to eight cores are given below in Figure 4.15.

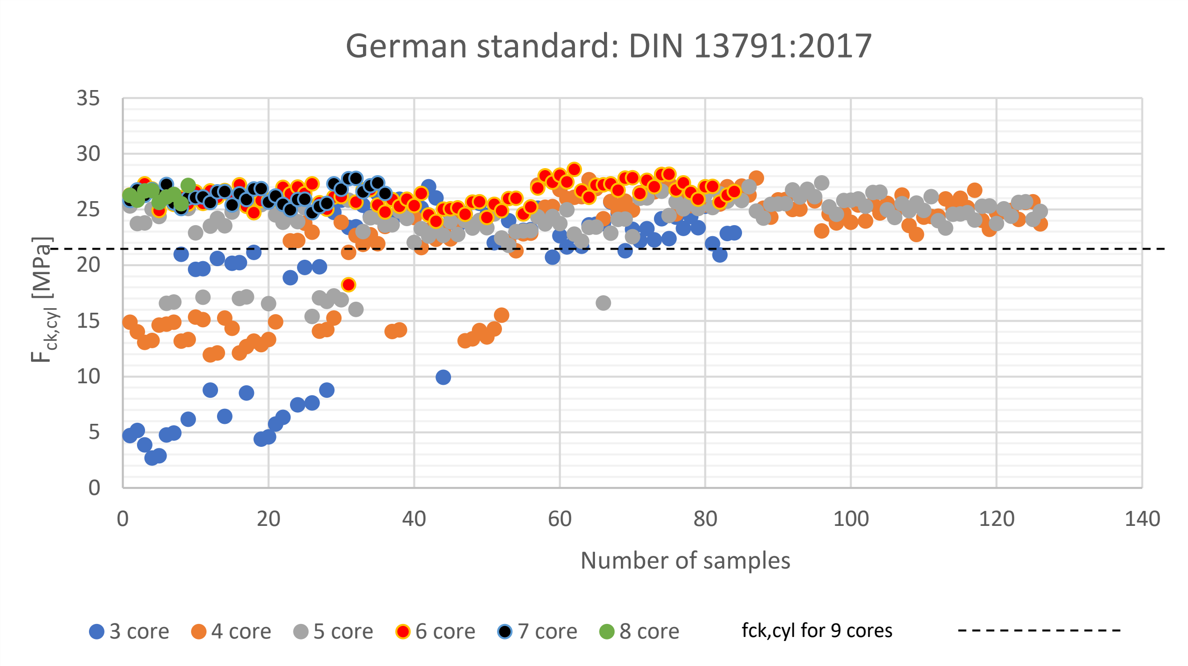


Figure 4.15: Results of three to eight cores according to DIN 13791:2017

The Netherland method generally results in conservative estimates. The estimates for three cores range from 12-26 MPa. The use of four to five cores results in 16-25 MPa. The results are shown below in *Figure 4.16*.

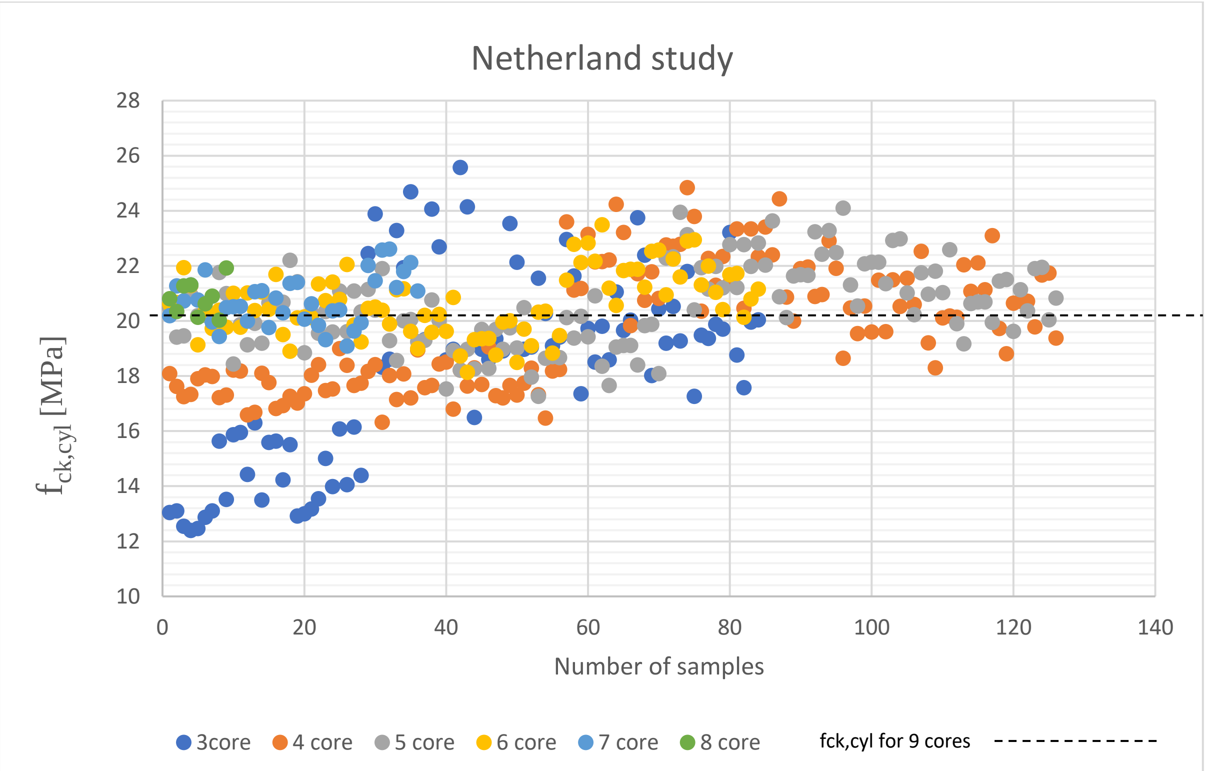


Figure 4.16: Results of three to eight cores according to the Netherland study.

The scatter of all the interpretation methods decreases with the increase number of cores. For number of cores of three and four, the scatter is approximately the same for approach B and TDOK at 11.6 MPa and 9.3 MPa respectively. The scatter for three cores obtained from the GCSC and Netherland study is 15 MPa and 13.2 MPa respectively. However, the scatter observed from the German study DIN 13791:2017, is much larger at 24.3 MPa for three cores and 16.2 MPa for four cores. The scatter of GCSC is large for one core at 14 MPa, for two cores at 9 MPa and for three cores at 10MPa. But a scatter of 5 MPa was obtained and constant from 5 core onwards. After 6 cores, the values from each method converges to small values.

Table 4.3: Scatter of estimation of compressive strength for different methods and number of cores

		Scatter [MPa]				
Number of cores	EN13791/ 2019, 8.1	EN13791/2007 Approach B	GCSC	TDOK	German	Netherland
1	NA	NA	14,0	NA	NA	NA
2	NA	NA	9,0	NA	NA	NA
3	NA	11,6	15,0	11,6	24,3	13,2
4	NA	9,3	10,0	9,3	16,2	8,5
5	NA	4,9	5,0	7,5	12,0	6,8
6	NA	5,8	5,0	4,0	10,3	5,4
7	NA	3,8	5,0	3,3	3,1	3,5
8	3,8	2,0	5,0	2,6	1,6	1,9

NA = Not applicable

Table 4.4. Comparison of different interpretation methods

	Advantages	Disadvantages
EN13791/2019 8.1	-Safer result for minimum number of cores 8 -For correlation it is economical it considers many classes of strength	-For small cores there is -volume limitation and overestimation of results -Possible low values for high C.V for number of cores larger than 8 -large number of pairs of data for correlation
EN13791/2007	-Practical, can be used for Small number of cores, practical -For correlation it is economical it considers many classes of strength	-Overestimation for high C.V and unsafe for structural assessment -Large number of pairs of data for correlation

GCSC	-Practical, quick, safe average results for several cores -Depends on probabilistic distribution -It gives additional information of concrete strength at the time of construction	-Underestimation or overestimation can occur for small number of cores -Safer for cores 4 and above
TDOK	- Small number of cores→Practical with respect	-Overestimation possibility for high C.V
Netherland study (EN 1990)	-Small number of cores→Practical with respect to economy -Safer result	-Low values of estimation for small number of cores
German standard DIN EN 13791:2017	-Practical with respect to economy -Safer result, can be used for small number of cores	-Can yield low result for high C. V -Few overestimations can occur

5 Discussion

5.1 Method of testing

The simplicity, low cost and widespread use of Schmidt hammer and UPV make those methods most suitable for the study. Those features of Schmidt hammer and UPV tests were crucial because there were limitations of time and resources. However, the methods are the most unreliable. As the assessment of the methods according to table 2.1.4 shows various factors affecting the results. The other methods which are affected by less factors and which measure the strength more directly such as pull-out and windsor probe are potential methods that can give more correct correlation. Less number of cores may be enough for these direct methods because they give more correct results. For methods with high variability such as schmidt hammer may need more cores for estimating the compressive strength.

It is also good to assign more correct methods for concrete suspected of high variability of quality, for instance as a result of poor-quality control. Direct methods should be applied on concrete suspected with higher variability of quality because it can give more accurate results. If indirect method such as Schmidt Hammer is used on a concrete with significant variation of quality, the variability of the results may become worse.

Results of Schmidt hammer depend on the slenderness of the tested elements, but the effect from slenderness is negligible for the elements tested in Skuru bridge. There is a carbonation depth of 30 mm in the Skuru bridge. However, the carbonation depth effect is taken care of the correlation with drilled cores. UPV results can be affected by the depth of the location of objects such as reinforcements and voids. Therefore, if the indirect test result is low that cannot be interpreted as that the quality of concrete is low at that particular location. The results show only that further investigation is necessary to determine the real cause of the low result.

At last adequate care is required for identifying the concrete location that can be considered a good measure of the strength. Hence, the study of the construction documents is vital in order to acquire prior knowledge about the construction. Moreover, it is helpful to undertake careful visual inspection of the concrete for planning of the tests.

5.2 Comparison of results

The estimations of the in-situ characteristic compressive strength using different methods listed in *Table 4.2* has shown scatter as presented in *Figure 4.9.* For EN 13791:2007 and EN 13791: 2019, there is difference in the results because the estimation procedures have changed. The changes are reflected in that the current procedures are aligned to EN 1990 which is based on 95% probability of the t-statistics. On the other hand, approach, A and B utilize different procedures of calculation. In the new standard confidence number (k_{n}) depends on the number of cores and is multiplied by the standard deviation. However, the formulas in approach A use a constant k_2=1.48 and in approach B the value of k is not multiplied by the standard deviation. Hence this procedural change has resulted ultimately in different values of estimation.

Literature studies also show that the estimation results obtained from EN 13791:2007 is higher than that of EN 13791:2019. The results of approach A are, however, lower. That is because the estimates are independent of the standard deviation of the sample in Approach B. and for the approach A the results are lower due to the value of k_2 =1.48. However, the estimates of the approaches do not represent the statistical uncertainties as accurately as the EN 13791:2019 and EN 1990.

In the case of EN 13791:2019, when only cores are used for estimation, the parameter k_n is used which depends on the number of cores. This parameter is adopted from the Table D.1. in EN 1990. Weber's study indicate that these formulations accurately consider the statistical variation and give safer result. Therefore, the application of the clause 8.1 is recommended for safe structural calculations if the number of cores is larger than eight. The results of the estimations for the Skuru bridge also yields lower results when the standard is used. The standard deviation of the Skuru bridge is 6.3 MPa. For this standard deviation the difference of characteristic compressive cylinder strength with EN 13791:2007 is about 5.6 MPa. If the standard deviation of the sample increases, the overestimation of EN 13791:2007 increases because it is independent of the standard deviation. Hence, for smaller standard deviation as in Skuru bridge, the overestimation may not be critical. However, for large values of standard deviations, the overestimation can be unsafe.

Furthermore, the estimates show higher scatter as the number of cores decreases as shown in *Figures 4.10-4.16*. It is unavoidable to have higher scatter of coefficient of variation as the number of cores becomes lesser. That is because as shown in Appendix 8.7 the scatter of the coefficient of variation becomes larger with decreased number of cores. Those higher scatter of coefficient of variation also introduce higher scatter of estimates.

The reason why there are differences between the results is because they are based on different methodologies. EN 13791:2019, the Netherland and DIN 13791:2017 are based on EN 1990. EN 1990 provides the statistical means of assessing uncertainties. Consequently, those methods yield safer result which can be conservative at times. On the other hand, EN 13791:2007, TDOK and the GCSC Method have different methodological basis that considers statistical uncertainties to a lesser degree. Furthermore, those methods have less reliability of the prediction.

As mentioned earlier, the estimates from EN 13791:2007, TDOK and GCSC may have been already overestimated because of the methodology. The estimation values are always larger as presented in *Figures 4.10-4.16* than EN 13791:2019 and Netherland values. Therefore, the overestimation problem can be worsened for small number of cores. It is better to use the Netherland method or the German standard particularly for high coefficient of variation of small number of cores since they have no methodology problem of overestimation.

In the estimation of in-situ characteristic compressive strength using the EN 13791:2007 for correlated pairs of data, the standard deviation considered is only for the estimates of in-situ compressive strength from the correlation. This results in estimate that does not accurately represent the actual spread in the core results. The new EN standard has improved estimation procedures for correlation that accounts for both the spread in the core results and the estimates from the correlation curve. Hence, the correlation from the updated 13791 standard results in values that are more accurate.

In the case of the correlation performed in the Skuru bridge, the number of cores is not adequate. This has resulted in negative correlation. Consequently, this negative correlation resulted in incorrect strength estimates. The estimate of the characteristic in-situ strength inherited this error and wrong results are obtained as shown in Appendix 8.1.5-8.1.7. Therefore, the use of small cores for correlation is unreliable.

The GCSC results obtained for five cores and above are between 25 MPa and 30 MPa, as shown in *Figure 4.10*. The method is developed from practical experience and the probabilistic considerations. Thus, the design of the method performs well in limiting high values and results most of time in similar values to TDOK and EN 13791:2007 for low coefficient of variation. However, some times the results could be conservative or overestimated for number of cores one to three. Hence, the methodology of the method can be improved for the use of one to three cores. Still, this probabilistic approach of the method is promising to be useable for estimation of characteristic in-situ compressive strength. However, the method may overestimate for samples with high coefficient of variation.

An advantage of this GCSC method is that this method depends not only on the number of cores, but also on the model distribution as shown in *Figure 2.56*. Additionally, the method could be applied when there is a demand of a rapid and rough estimation of compressive strength. For one to two cores the result for the design CSC presents a large scatter because it varies with the core drill results as shown in *Figure 4.10*. Further, using three cores also results in overestimation or underestimation of CSC results. Hence, in these cases the use of supplementary methods or the increase of number of cores is recommended.

Another advantage with the GCSC method is that the method provides a way of estimating the CSC at the time of construction. It also provides information about the age hardening effects and recommended design CSC of the concrete depending on the environment of the structure.

Generally, the use of the interpretation methods should be based on the assumptions and the limitations of each method. The methods result could need engineering judgment and scientific methodology assessment particularly for small number of cores. For instance, the results of EN 13791:2007, TDOK and GCSC method respond to variation to lesser degree than that of EN 1990 based methods as shown in Appendix 8.6. The variation is not considered to the same degree as the EN 13791:2019, Netherland method and DIN 13791:2019. For high coefficient variation DIN 13791:2017 result in lower estimate. As the number of cores increase the variation also decreases. Therefore, the increased number of cores can in turn increase the estimate of the method.

Therefore, the use of the interpretation methods should be reflected on how much accuracy, safety and certainty is needed. If the variation is low or the certainty is not that much important, the methods that depend less on the variation may be used for assessment of the in-situ characteristic compressive strength. However, if there is much uncertainty about the concrete, the methods that put emphasis on the uncertainties are preferable. For example, the Netherland and DIN EN 13791:2017 limit the overestimation for low coefficient of variation and give safer result.

The interpretation methods have their own merits and demerits as presented in *Table 4.4* and those aspects of the methods are necessary in assessments. Hence, there is no wrong or incorrect result that is obtained if the assumptions, the purpose and the individual judgment is considered. However, the correct judgment about the statistical uncertainty and the characteristics of the concrete under investigation is decisive. The use of averaging methods or probabilistic methods may be acceptable to concrete which is relatively new, for number of cores that are significantly larger or when other inspection confirms sound concrete. However, if the concrete is very old and visual inspection shows some signs of distress, the statistical uncertainties need more exact assessment.

5.3 Recommendations

The Pull-out test according to *Table 2.3* is affected by few factors. The method also provides better correlation to compressive strength compared to the schmidt Hammer and UPV. Therefore, this method is highly recommended to be applied if it is possible. There are also guidelines in the Eurocode that describe the application and procedures.

Further, it is recommended to adopt the use of windsor penetration method which has a widespread use in USA. This method is affected by few factors and results in good accuracy of estimation. Additionally, it does not damage the concrete. However, its application is limited in the European area since there is no European standard of guidance.

The EN 13791:2019, clause 8.3 offers a method for estimation of in-situ characteristic compressive strength for number of cores three to seven. However, the method requires limitations of volume and spread of the core results. The practical applications have shortcomings and the criteria of spread is found to be questionable. According the study by Rabea Sefrin, the method also results in overestimation. If safer result is critical for smaller number of cores, the Netherlands method and the DIN EN 13791:2017 can be utilized which have better reliability.

According to RILEM appropriate statistical procedures need to be considered in the determination of characteristics compressive strength. The methods involving only averaging of values such as Clause 8.3 of 13791:2019 may not result in accurate estimates.

In addition, if the number of cores tested is small or a small area is covered in the investigation, the standard deviation could be low. It is therefore recommended to set a minimum standard deviation as per the Netherlands study (Vervuurt et al. 2013) to obtain safer results.

5.4 Possible future study

A possible study that may partly solve the difficulty of reliable results with the use of small number cores is to estimate the compressive strength of structural members from large set of data of similar structures (Vervuurt et al. 2013). Therefore, this type of study could be developed by analyzing a large set of bridges and assessing the compressive strengths and use the available data for assessment of any other bridge.

The use of correlation in this study had limitation of time and resource. Hence, the results are not reliable since the number of cores did not met the minimum criteria set by the standards. The study can be expanded to assess the results of correlation by using large number of data pairs.

The results of the GCSC method have shown consistent results for cores more than three. Further study could be possible in enhancing the accuracy for smaller number of cores.

6 Conclusion

To conclude, there is difference in the scatter of the various methods for estimating the in-situ characteristic compressive strength. Generally, the problem of accuracy worsens with smaller number of cores. The standard methods of EN 13791:2019 and EN 1990 result in conservative values for minimum of eight cores, since the methods consider more precisely the statistical spread of the core results. The other interpretation methods also perform better for lower coefficient of variation of samples. Correlation is suitable in case of large structures consisting different concrete classes because it reduces the number of cores required to meet the standards criteria.

For 9 core test results, there is convergence of the estimates at 8 cores. The CSC for TDOK and EN 2007 are C30/37. On the other hand, the CSC for EN 13791 2019 is C20/25. The scatter of the estimates is due to the methodology of the interpretation methods.

7 References

Adam C. Carroll; Aaron R. Grubbs; Anton K. Schindler; Robert W. Barnes. 2016. “Effect of core geometry and size on concrete compressive strength”, The Alabama Department of Transportation, pp 5-9

American standard: ASTM C1583/C1583M. 2020. “Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)”, pp 1-4

American standard: ASTM C597-16. 2016. “Standard Test Method for Pulse Velocity Through Concrete”, pp 2-4

American standard: ASTM C900-19. 2020. “Standard Test Method for Pullout Strength of Hardened Concrete” pp 1-3

American standard: ASTM C900-19: “Standard Test Method for Pullout Strength of Hardened Concrete”, 2020, pp 1-3

Ariöz Ö., Tuncan A., Tuncan M., Kavas T., Ramyar K., Kilinic K. and Karasu B. 2009. “Use of combined non-destructive methods to assess the strength of concrete structures” pp 147-148

A.T Moczko ., N.J Carino, and C.G Petersen. 2016. “CAPO-TEST to Estimate Concrete Strength in Bridges”, ACI MATERIALS JOURNAL, pp 827-836

Latte Bovio. F.; Latte Bovio. G.; Brencich. A.; Cassini. G.; Pera. D; Riotto. G. 2014. “A New Pull-Out Technique for In-Place Estimation of Concrete Compressive Strength”, pp 2-3

Bonora. V.; Carli. R.; Di Leo. A.; Giovanni. 2000. “Evaluation of Actual Compressive Strength of High Strength Concrete by NDT”

Branco A.F.; De Brito J. 2004. “Handbook of concrete bridge management”, pp 40

Brencich. A.;Bovolenta. R. 2020.” Rebound Hammer Test: An Investigation into Its Reliability in Applications on Concrete Structures”

Breysse D. 2012. “Non-destructive assessment of concrete structures: Reliability and Limits of single and combined techniques”, pp 17-21; 119-182; 110-127; 128-141

Brožovský. J.” Evaluation of calculation for correlation efficiency as mentioned in 13791 in order to determine compressive strength by nondestructive testing)

Chakiri, H. “Master’s thesis: -Appraisal of the compressive strength for concrete in situ”

Dyson J. et al, 2017.” Proposed Recommended Practice on cores and NDT for concrete strength assessment – A replacement for CIA Z11”

Eduardo. P.; Marcelo. H. 2012. “Pull Off test to evaluate the compressive strength of concrete: an alternative to Brazilian standard techniques”, vol. 5, pp 760-761

Ergün. A.; Kürklü. G. 2012. “Assessing the Relationship between the Compressive Strength of Concrete Cores and Molded Specimens”, Gazi University Journal of Science, pp 737

European Standard: EN 12504-4:2004. 2004. “Testing concrete – Part 4: Determination of ultrasonic pulse velocity”, pp 4-12

European Standard: EN 12504-3. 2005. “Testing concrete in structures – Part 3: Determination of pull-out force”, pp 5-9

European standard: EN 13791:2019. 2019. “Assessment of in-situ compressive strength in structures and precast concrete components.

European Standard EN 12504-1. 2019. “Testing concrete in structures – Part 1: Cored specimens – Taking, examining and testing in compression”, pp 3-9

European Standard EN 12390-1. 2019. “Testing hardened concrete – Part 3: Compressive strength of test specimens”, pp 5-6

European Standard EN 13791. 2019.” Assessment of in-situ compressive strength in structures and precast concrete components”, pp 21

H.A Mohammed Bishr. 1990. “Assessment of concrete strength by lok and capo tests”, Faculty of the college of graduate studies king Fahad University of petroleum & minerals, Dharan Saudi Arabia, pp 20-22

Harrison T. 2015. “Assessment of concrete compressive strength in structures”

Helmerich R.; Baessler R.; Buhr B.; Holm G.; Krüger M.; Niederleithinger E.; Tomor A.; Brühwiler E.; Bernardini G.; Phares B. 2007. “Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives”, pp 37

John H. Bungey; Stephen G. Milliard; Michael G. Grantham. 2006.” Testing of concrete structures”, 4th edition, pp 36-139; 51-62; 111-116; 93-103

Karahan Ş.; Büyüksaraç A.; Işık E. 2020. “The Relationship Between Concrete Strengths Obtained by Destructive and Non-destructive Methods”, pp 2-6

M.F. Abbas Alwash. 2017. “Assessment of concrete strength in existing structures using nondestructive tests and cores: analysis of current methodology and recommendations for more reliable assessment”, pp 43-48

Naderi. M. 2006. “Assessing the in situ strength of concrete, using new twist-off method”, Vol. 4, pp 147

Proceq. (2017). “Pundit® Portable Ultrasonic Pulse Velocity Testing”, pp 2-3 PowerPoint Presentation (proceq.com) (Retrieved 2021-02-11)

RILEM Technical Committee 249-ISC, https://www.rilem.net/groupe/249-isc-non-destructive-in-situ-strength-assessment-of-concrete-295, retrieved on 13-3-20121

Sadzevicius R.;Sankauskiene T.; Milius P. 2015. ” Comparison of concrete compressive strength values obtained using rebound hammer and drilled core specimens”, Rural development, pp 1

Sangiorgio F. 2018. "A graphical method for assessing the concrete strength class in existing RC structures." *Materials and Structures*, *51*(3), pp.1-22.

Sefrin R.; Glock. C.; Schwabach. E. 2020. "Determination of the characteristic compressive strength of concrete in existing structures according to DIN EN 13791: 2020 for small sample sizes"

Schabowicz. K. ;Sterniuk. A.; Kwiecińska. A. ;Cerba. P. 2018. "Comparative analysis of selected non-destructive methods for concrete diagnosis", pp 3

Sitorus. T.; Jaya. I. 2020. "Evaluation of compressive strength investigation on job mix test object and core drill test object", Department of Civil Engineering, Faculty of Engineering, Universitas Sumatera Utara, Medan, Indonesia, pp 1-3

Trtnik G.; Kavcic F.; Turk G. 2008. "Prediction of concrete strength using ultrasonic pulse velocity and artificial neural networks", pp 53

VERVUURT. A.; Courage. W.; Steenbergen. R. 2013. " Strength characteristics for the structural assessment of existing concrete Structures", Netherlands.

V.M. Malhotra; N.J Carino. 2004. "handbook of non-destructive testing of concrete", 2nd edition.

Trafikverket. 2020. "Bärighetsberäkning av broar: TDOK 2013:0267", version 7.0, pp 27-28

Trafikverket. 2019. "Krav Brounderhåll: TDOK 2013:0415", version 4.0, pp 28.

Weber M. 2019. "Zur Ermittlung mechanischer Kennwerte von Bestandsbetonen"

8 Appendices

8.1 Detail calculations of estimation of characteristic in situ compressive strength

8.1.1 Calculations according to TDOK

Calculation of compressive strength according to TDOK 2013:0267 for six samples or more with core strength test of 100 mm diameter 1:1 cores.

The in-situ strength of the concrete for 9 test locations are given in the table below.

Test location	**4N-1**	**4N-2**	**4N-3**	**5N-1**	**Arch1**	**Arch2**	**26N-1**	**26N-2**	**26N-3**
$F_{c\ 1:1}$ core	28.1	47.1	37.4	40.2	45.8	33.4	33.7	43.4	38.3
$f_{c,is}$	23.0	38.6	30.7	33.0	37.6	27.4	27.6	35.6	31.4
x+5	33,1	52,1	42,4	45,2	50,8	38,4	38,7	48,4	43,3
x/0,8	35,125	58,875	46,75	50,25	57,25	41,75	42,125	54,25	47,875

$m = 38.6$ Average of the in-situ strength results

$s = 5.12$ Standard deviation of the in-situ strength results,

$x = strength\ value\ for\ individual\ sample$

$m \geq f_{\mathrm{KK}} \rightarrow f_{\mathrm{KK}} \exp\left(\frac{1{,}4s}{m}\right) = \mathbf{30,77\ MPa}$

$x \geq f_{\mathrm{KK}} - 5 \rightarrow f_{\mathrm{KK}} = OK$

$x \geq 0{,}8 f_{\mathrm{KK}} \rightarrow f_{\mathrm{KK}} = OK$

The obtained K-value is K35 which corresponds to **C30/37**

8.1.2 Calculations according to EN 13791:2007

Calculation of compressive strength according to 7.3 of SS EN 13791:2007 for 3 to 13 valid results of core strength test of 100 mm diameter 1:1 core.

The in-situ strength of the concrete 9 test locations is given in the table below.

Test location	**4N-1**	**4N-2**	**4N-3**	**5N-1**	**Arch 1**	**Arch 2**	**26N-1**	**26N-2**	**26N-3**
$F_{c\ 1:1}$ core	28.1	47.1	37.4	40.2	45.8	33.4	33.7	43.4	38.3
$f_{c,is}$	23.0	38.6	30.7	33.0	37.6	27.4	27.6	35.6	31.4

$f_{\mathrm{c,m(n),is}} = 38.6$ Average of the in-situ strength results

$S_{\mathrm{s}} = 5.12$ Standard deviation of the in-situ strength results,

$K = 6$ From *Table 2* of EN 13791:2007

$f_{\mathrm{ck,is}} = f_{\mathrm{c,m(n),is}} - K = 32{,}6\ \mathrm{MPa}$ (1)

$f_{\mathrm{ck,is}} = f_{\mathrm{c,is,lowest}} + 4 = 32{,}1\ \mathrm{MPa}$ (2)

The characteristic strength of the in-situ concrete is the lowest of 1 and 2,

$f_{\mathrm{ck,is}} = 32{,}1$ MPa, the concrete class is therefore **C30/37**

8.1.3 Calculations according to EN 13791:2019

Calculation of compressive strength according to clause 8.1 of SS EN 13791:2019 for 9 valid results of core strength test of 100 mm diameter 1:1 core.

The in-situ strength of the concrete 9 test locations is given in table below:

Test location	**4N-1**	**4N-2**	**4N-3**	**5N-1**	**Arch1**	**Arch2**	**26N-1**	**26N-2**	**26N-3**
$f_{\mathrm{c,1:1\ core}}$	28.1	47.1	37.4	40.2	45.8	33.4	33.7	43.4	38.3
$f_{\mathrm{c},is}$	23.0	38.6	30.7	33.0	37.6	27.4	27.6	35.6	31.4

$f_{\mathrm{c,m(n),is}} = 31.7$, Average of the in-situ strength results

$S_{\mathrm{s}} = 5.12$, Standard deviation of the in-situ strength results,

$S_{\mathrm{s}} \geq 0.08 \cdot f_{\mathrm{c,m(n),is}} = 0.08 \cdot 31.7 = 2.53$, C. V=8%

$k_{\mathrm{n}} = 2$ From Table 6 of EN 13791:2019

$f_{\mathrm{c,is,ck}} = f_{\mathrm{c,m(n),is}} - k_{\mathrm{n}} \cdot S_{\mathrm{s}} = 31.7 - 2 * 5.12 = 21.7\ \mathrm{MPa}$ (1)

$f_{\mathrm{c,is,ck}} = f_{\mathrm{c,is,lowest}} - M = 23 + 4 = 27$, M from table 7 of EN 13791:2019 (2)

The characteristic strength of the in-situ concrete is the lowest of 1 and 2,

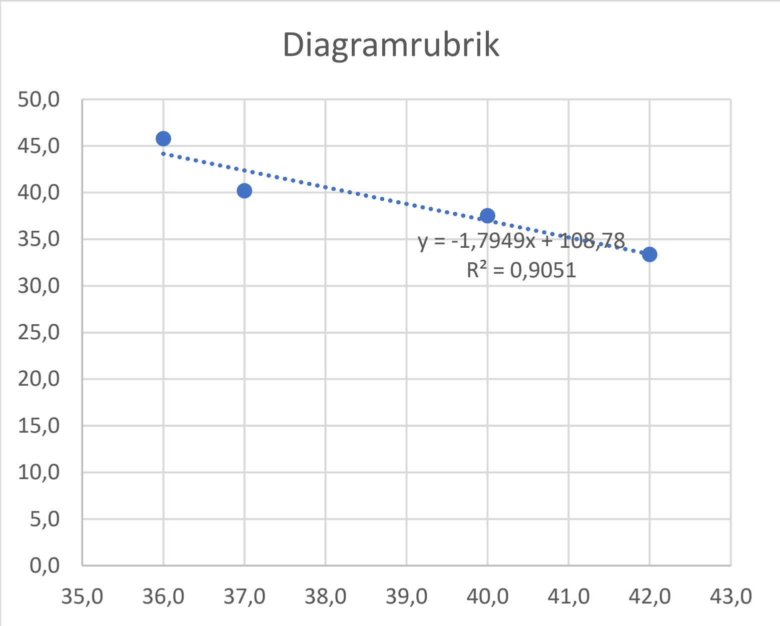
$f_{\mathrm{c,is,ck}} = 21$ MPa, the concrete class is C20/25

8.1.4 Calculations according to EN 13791:2007, Alternative 1 for Rebound Number

Calculation of compressive strength according to use of EN 2007 for correlation of 4 valid pairs of results of core strength test of 100 mm diameter 1:1 cores and RN

1) Establishing of best fitting curve

Test location	RN	$\mathbf{f_{c,is,cube}}$	$\mathbf{f_{c,is,cyl}}$
Column4N	40.0	37.5	30.8
Column5N	37.0	40.2	33.0
Arch1	36.0	45.8	37.6
Arch2	42.0	33.4	27.4



2) Estimation of the characteristic in situ compressive strength

Test location	**R**	$\mathbf{f_{is,l}}$
Column 4N	40	36.98
Column 5N	37	42.37
Column 4S	41	35.19
Column 5S	35	45.96
1stArch	36	44.16
2stArch	42	33.39
Column27N	50	19.03
Column27S	38	40.57

$f_{\mathrm{m(n)},is} = 37.2$ MPa $s \geq 3$ MPa

$s = 8.5$ MPa,

$$f_{\mathrm{c,is,ck}} = f_{\mathrm{m}(n),is} - 1.48 \cdot \mathrm{s} = 24.54 \text{ MPa}$$

$$f_{\mathrm{c,is,ck}} = f_{\mathrm{c},is,lowest} + 4 = 23.03 \text{ MPa}$$

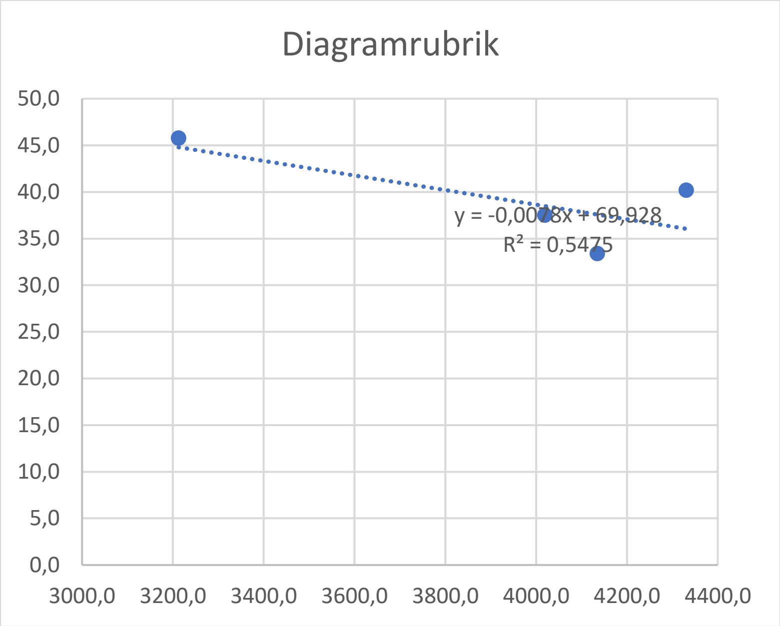
Hence, the CSC is C20/25 according to EN 206-1

8.1.5 Calculations according to EN 13791:2007, Alternative 1 for UPV

Calculation of compressive strength according to use of EN 2007 for correlation of 4 valid pairs of results of core strength test of 100 mm diameter 1:1 core and UPV

1) Establishing of correlation curve

Test location	UPV	$f_{c,is,cube}$
Column4N	4019.0	37.5
Column5N	4330.5	40.2
Arch1	3212.7	45.8
Arch2	4134.5	33.4



2) Estimation of characteristic in situ compressive strength

Test location	**UPV**	$f_{is,l}$
Column 4N	4019	38.58
Column 5N	4331	36.15
1stArch	3213	44.87
2stArch	4135	37.68
Column27N	4536	34.55
Column27S	4748	32.90

$f_{\mathrm{m(n)},is} = 37.4 \text{ MPa}$

$s = 5.2 \text{ MPa}, \qquad s \geq 3 \text{ MPa}$

$f_{\mathrm{c,is,ck}} = f_{\mathrm{m}(n),is} - 1.48 \cdot \mathrm{s} = 29.8 \text{ MPa}$

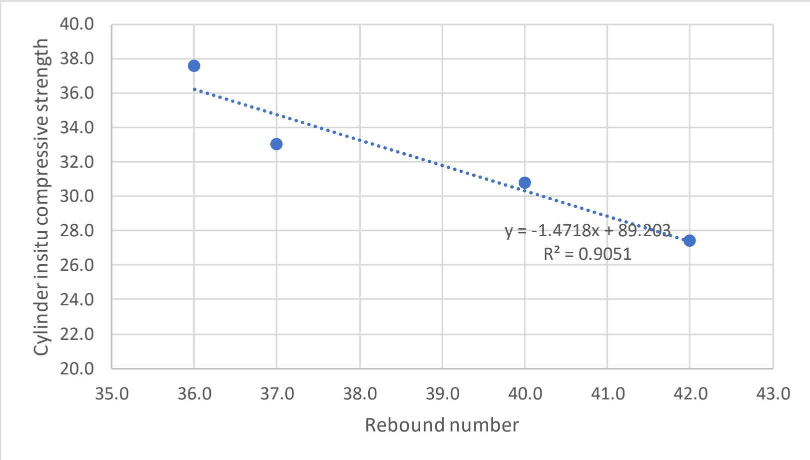
$f_{\mathrm{c,is,ck}} = f_{c,is,lowest} + 4 = 36.9 \text{ MPa}$

Hence, the CSC is C30/37 according to EN 206-1

8.1.6 Calculations according to EN 13791:2019 for correlation using Rebound hammer

Correlation of core insitu results with rebound number results according to SS EN 13791:2019 for 4 pairs of data. Minimum requirement of pairs of data is 10. Hence, this results has accuracy problem because of insufficient pair of data.

Test location	**R**	$f_{c,is,cube}$ **(MPa)**	$f_{c,is,cyl}$ **(MPa)**
Column4N	40.0	37.5	30.8
Column5N	37.0	40.2	33.0
Arch1	36.0	45.8	37.6
Arch2	42.0	33.4	27.4



Test location	$f_{c,reg}$ **(MPa)**	$(f_{c,reg} - f_{c,is,cyl})^2$
Column4N	29.81	0.9
Column5N	34.74	3.1
Arch1	36.21	1.8
Arch2	27.38	0.0
	Sc	**1.7**

Test location	**R**	$f_{c,reg}$ **(MPa)**	$f_{c,m,m}$ **(MPa)**	$(f_{c,reg} - f_{c,m,m})^2$
Column 4N	40	29.81	30.44	0.4
Column 5N	37	34.74	30.44	18.5
Column 4S	41	28.85	30.44	2.5
Column 5S	35	37.68	30.44	52.4
Arch1	36	36.21	30.44	33.3
Arch2	42	27.38	30.44	9.4
Column27N	50	15.60	30.44	**220.2**
Column27S	38	33.26	30.44	8.0
			Se	**7.0**

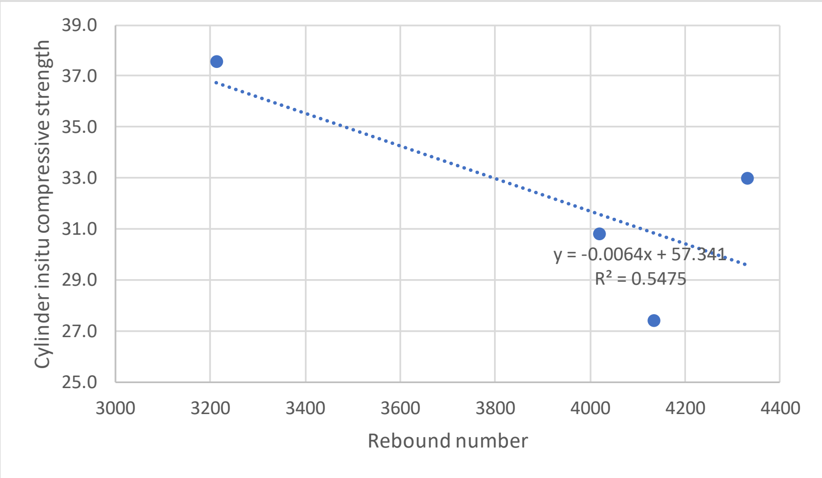
$f_{c,m(m),is}$	30.4
s	7.2
$n_{eff} + 1$	8.8
k_n	1.8
$f_{c,m(m)is} - k_n \cdot s$	17.5
$f_{c,i,lowest} + M$	**17.6**

CSC is C15/20

8.1.7 Calculations according to EN 13791:2019 for correlation using UPV

Correlation of core insitu results with Ultra pulse velocity results according to SS EN 13791:2019 for 4 pairs of data. Minimum requirement of pairs of data is 10. Hence, this results has accuracy problem because of insufficient pair of data.

Test location	UPV	$f_{c,is,cube}$(MPa)	$f_{c,is,cyl}$ (MPa)
Column4N	4019	37.5	30.8
Column5N	4331	40.2	33.0
Arch1	3213	45.8	37.6
Arch2	4135	33.4	27.4



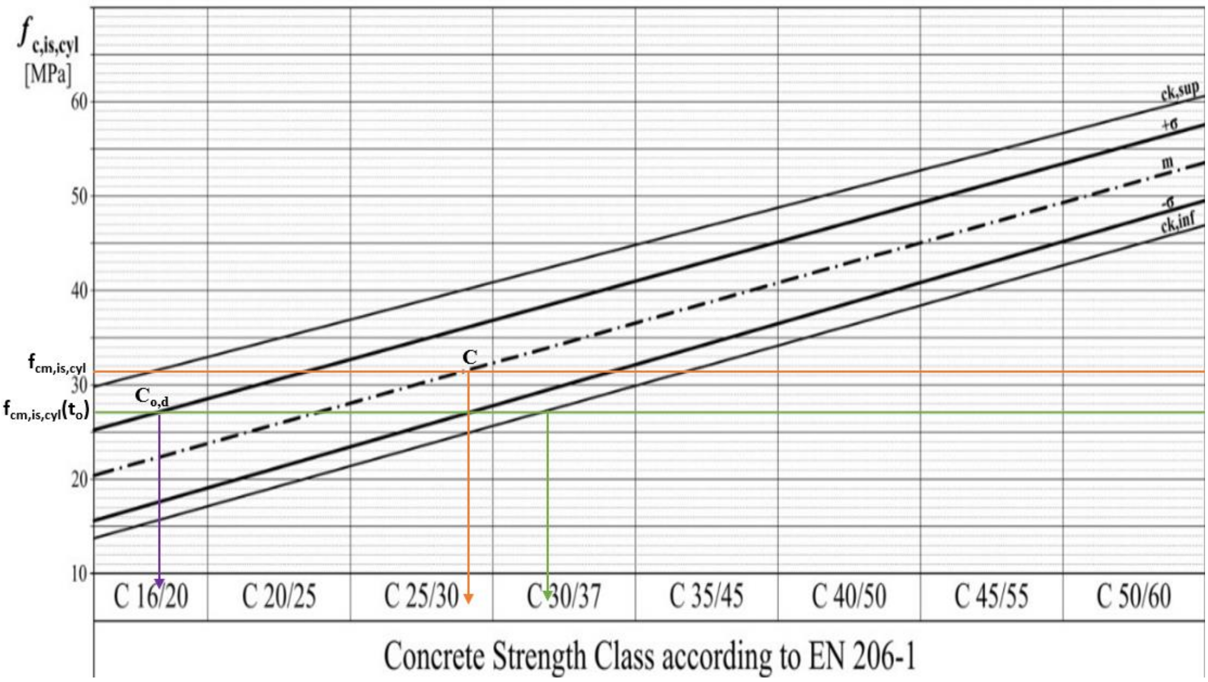
Test location	$f_{c,reg}$ (MPa)	$(f_{c,reg} - f_{c,is,cyl})^2$
Column4N	31.62	0.7
Column5N	29.62	11.2
Arch1	36.78	0.6
Arch2	30.88	12.2
	Sc	3.5

Test location	UPV	$f_{c,reg}$	$f_{c,m,m}$	$(f_{c,reg} - f_{c,m,m})^2$
Column 4N	4019	31.62	30.69	0.9
Column 5N	4331	29.62	30.69	1.1
Arch1	3213	36.78	30.69	37.0
Arch2	4135	30.88	30.69	0.0
Column27N	4536	28.31	30.69	5.7
Column27S	4748	26.96	30.69	14.0
			Se	3.4

$f_{c,m(m),is}$	30.7	
s	4.9	
$n_{eff} + 1$	6.6	
k_n	1.8	
$f_{c,m(n)is} - k_n \cdot s$	**21.9**	CSC is C20/25
$f_{c,i,lowest} + M$	31.0	

8.1.8 Estimation of CSC according to the graphical method

Test location	$f_{ck,cube}$	$f_{ck,is,cyl}$
Column4N	28.1	23.0
	47.1	38.6
	37.4	30.7
Column 5N	40.2	33.0
Arch1	45.8	37.6
Arch2	33.4	27.4
Pelarenära27N	33.7	27.6
	43.4	35.6
	38.3	31.4
$f_{cm,is,cyl}$	31.65	
$f_{cm,is,cyl}(t_o)$	27.00	



The CSC is determined by drawing horizontal line on the graph at $f_{\mathrm{cm,is,cyl}}$ and $f_{\mathrm{cm,is,cyl}}(t_{\mathrm{o}})$.

Therefore,

The CSC is C25/30 according to the table below.

The CSC at the construction time can be appraised from the cylinder strength at the time of the construction.

According to Table below for several cores:

$C_{\mathrm{inf}} \leq C_{\mathrm{o}} \leq C_{\mathrm{sup}}, \quad C16/20 \leq C_{\mathrm{o}} \leq C30/37$

$C_{\mathrm{o,d}} = \min\{C_{0,70}\} = \mathrm{C}16/20$

Table 5 Assessment of C, C_d, C_0 and $C_{0,d}$

Number of drilling cores	C and C_d	C_0 and $C_{0,d}$
One	$C_{inf} \le C \le C_{sup}$	$\begin{cases} C_{0,inf} \le C_0 \le C_{0,sup} & \text{for significant/moderate AHE} \\ C_0 = C & \text{for negligible AHE} \end{cases}$
	$C_d = \min\{C_{70}\}$	$C_{0,d} = \begin{cases} \min\{C_{0,70}\} & \text{for significant AHE} \\ C_{0,m} & \text{for moderate AHE} \\ C_d & \text{for negligible AHE} \end{cases}$
Two	$\max\{C_{inf,i}\} \le C \le \min\{C_{sup,i}\}$	$\begin{cases} \max\{C_{0,inf,i}\} \le C_0 \le \min\{C_{0,sup,i}\} & \text{for significant/moderate AHE} \\ C_0 = C & \text{for negligible AHE} \end{cases}$
	$C_d = \max\{C_{inf,i}\}$	$C_{0,d} = \begin{cases} \max\{C_{0,infi,i}\} & \text{for significant AHE} \\ \max\{C_{0,m,i}\} & \text{for moderate AHE} \\ C_d & \text{for negligible AHE} \end{cases}$
Several	$C = C_m$	$\begin{cases} C_{0,inf} \le C_0 \le C_{0,sup} & \text{for significant/moderate AHE} \\ C_0 = C & \text{for negligible AHE} \end{cases}$
		$C_{0,d} = \begin{cases} \min\{C_{0,70}\} & \text{for significant AHE} \\ C_{0,m} & \text{for moderate AHE} \end{cases}$

8.1.9 Estimation of CSC according to Netherland method

Calculations of characteristic in-situ compressive strength

#	Element	$f_{c,is,cube}$	$f_{c,is,cube}(Y)$
1	Column4N	28.1	3.33577
2		47.1	3.852273
3		37.4	3.621671
4	Column 5N	40.2	3.693867
5	Arch1	45.8	3.824284
6	Arch2	33.4	3.508556
7	Pelarenära27N	33.7	3.517498
8		43.4	3.770459
9		38.3	3.64545

$f_{c,m,cube}$	38.6
$f_{c,m,cube}(Y)$	3.64
S(Y)	0.167
n	9
t_{n-1}	1.86
s_{min}	10
s_{min}(Y)	0.255

$$s_{min}(Y) = \sqrt{\ln\left(1 + \left(\frac{s_{min}}{f_{cm,cube}}\right)^2\right)} = \sqrt{\ln\left(1 + \left(\frac{10}{f_{cm,cube}}\right)^2\right)} = 0.255$$

Method A:
$$f_{ck,cube} = \exp\{f_{cm,cube}(Y)\}.\exp\left\{-t_{n-1}(p = 0{,}05) \cdot s(Y) \cdot \sqrt{1 + \frac{1}{n}}\right\}$$

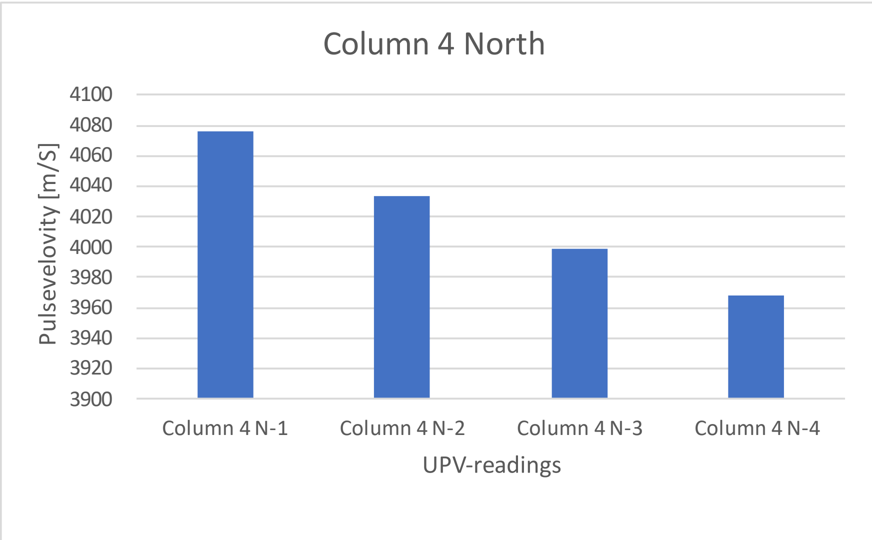
$$= \exp\{3.64\}.\exp\left\{-1.86 \cdot 0.167 \cdot \sqrt{1 + \frac{1}{9}}\right\}$$

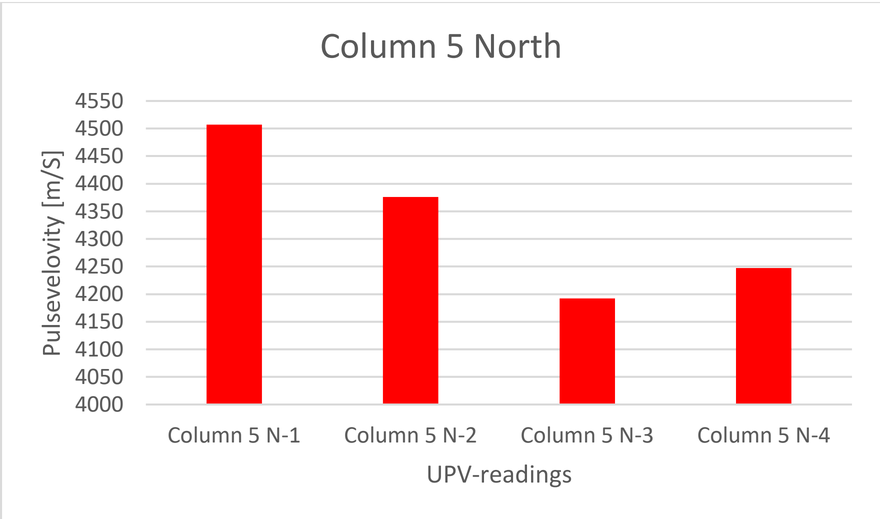
Method B: $$f_{\mathrm{ck,cube}} = \exp\{f_{\mathrm{cm,cube}}(Y)\}.\exp\left\{-1.64 \cdot s_{\min}(Y) \cdot \sqrt{1+\frac{1}{n}}\right\} = 27.46$$

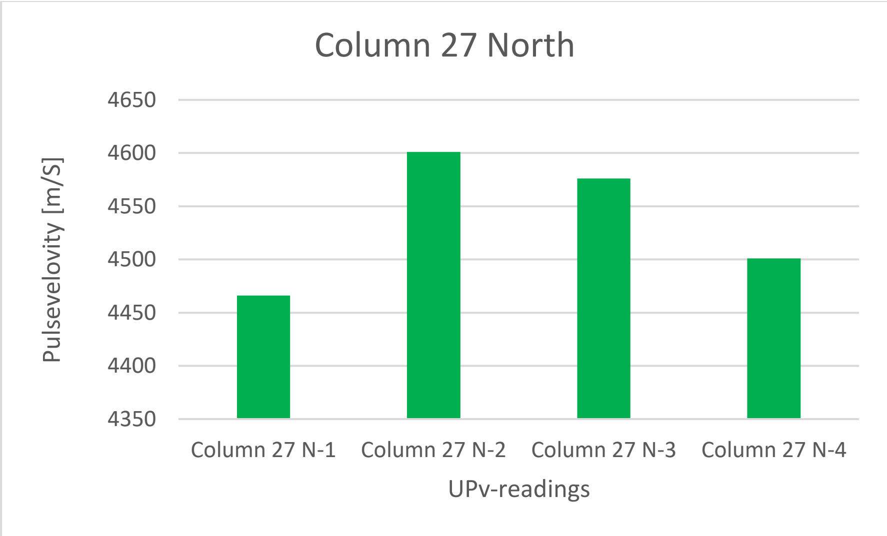
$$= \exp\{3.64\}.\exp\left\{-1.64 \cdot 0.255 \cdot \sqrt{1+\frac{1}{9}}\right\} = 24.54$$

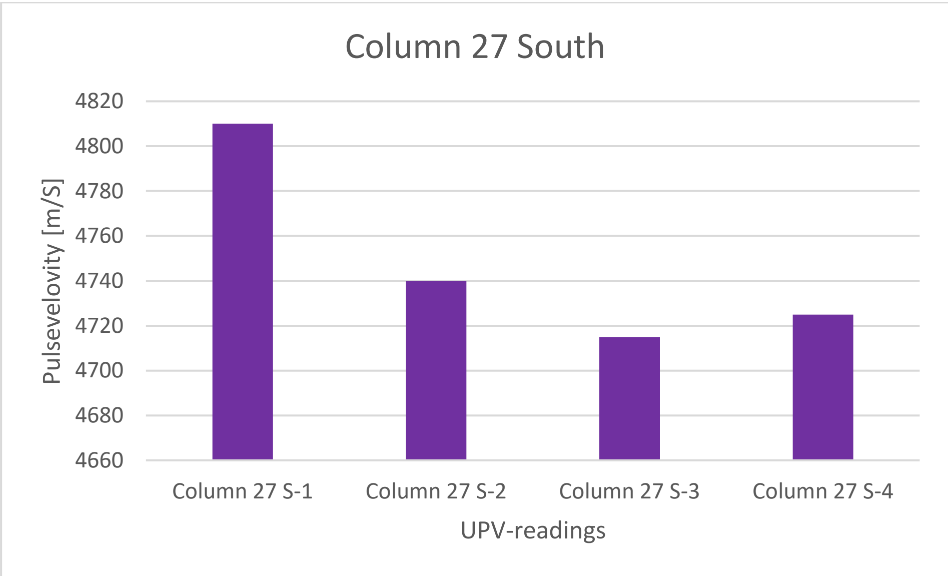
Therefore, $f_{\mathrm{ck,cube}} = 24.54$, Therefore the concrete strength class is C25/30

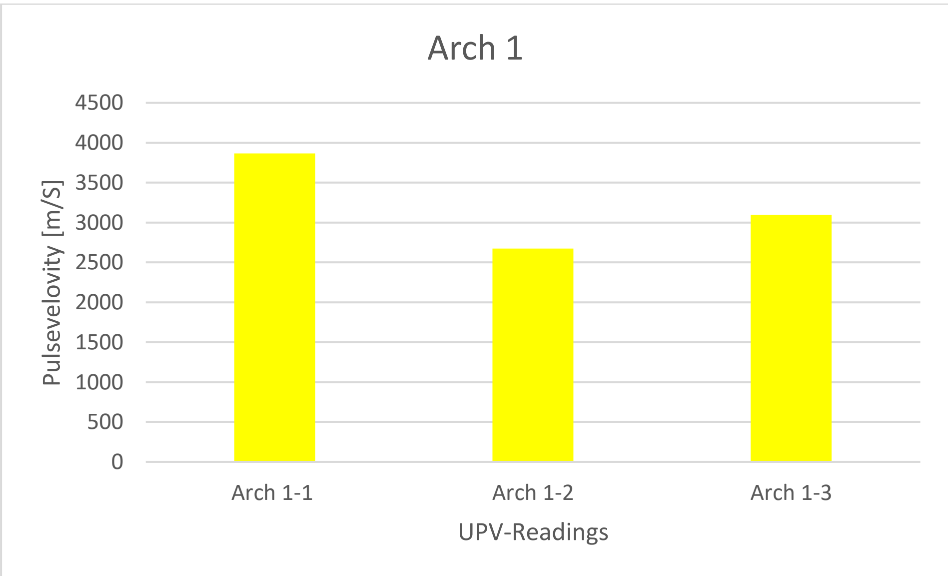
8.2 UPV-results

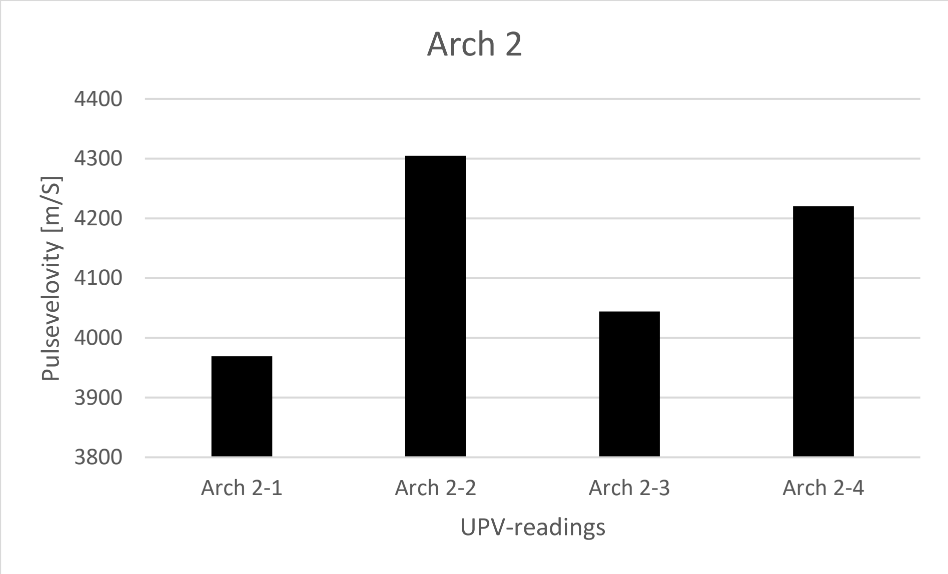




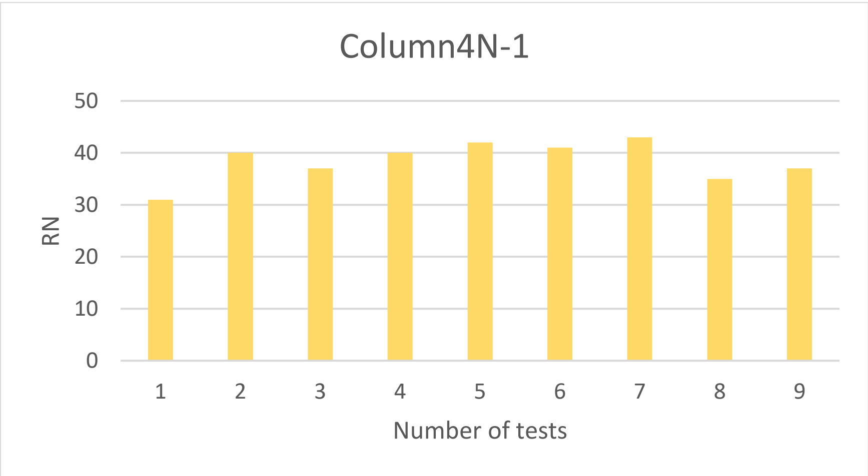

Column 27 North
4650
4600
4550
4500
4450
4400
4350
Pulsevelovity [m/S]
Column 27 N-1
Column 27 N-2
Column 27 N-3
Column 27 N-4
UPv-readings

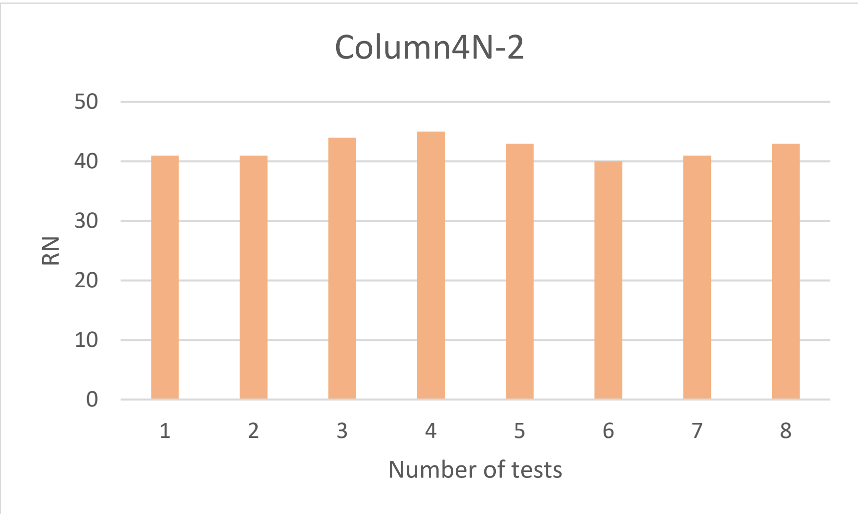

Column 27 South
4820
4800
4780
4760
4740
4720
4700
4680
4660
Pulsevelovity [m/S]
Column 27 S-1
Column 27 S-2
Column 27 S-3
Column 27 S-4
UPV-readings

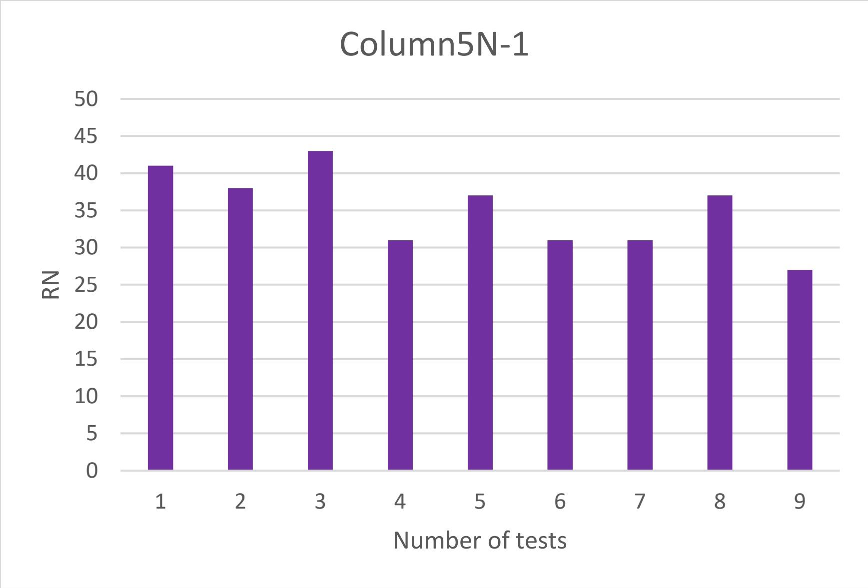

Arch 1
4500
4000
3500
3000
2500
2000
1500
1000
500
0
Pulsevelovity [m/S]
Arch 1-1
Arch 1-2
Arch 1-3
UPV-Readings



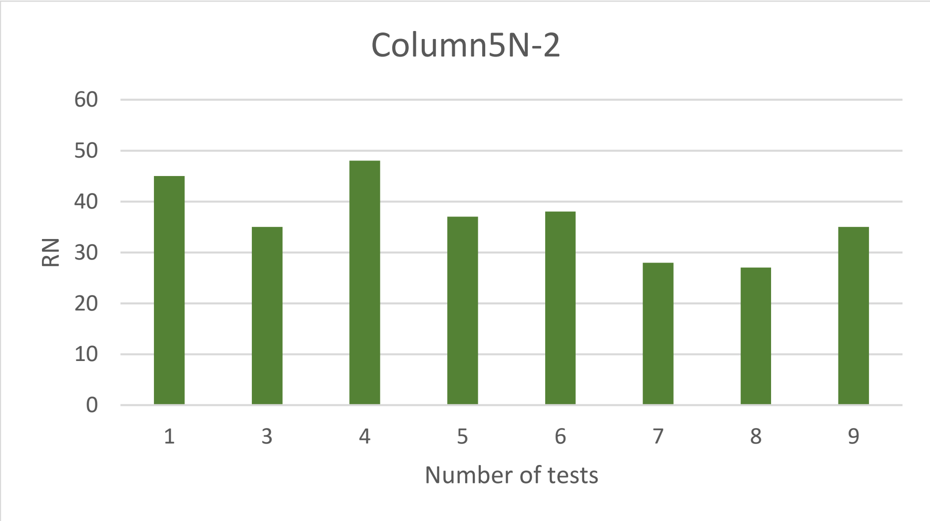
8.3 Schmidt hammer results



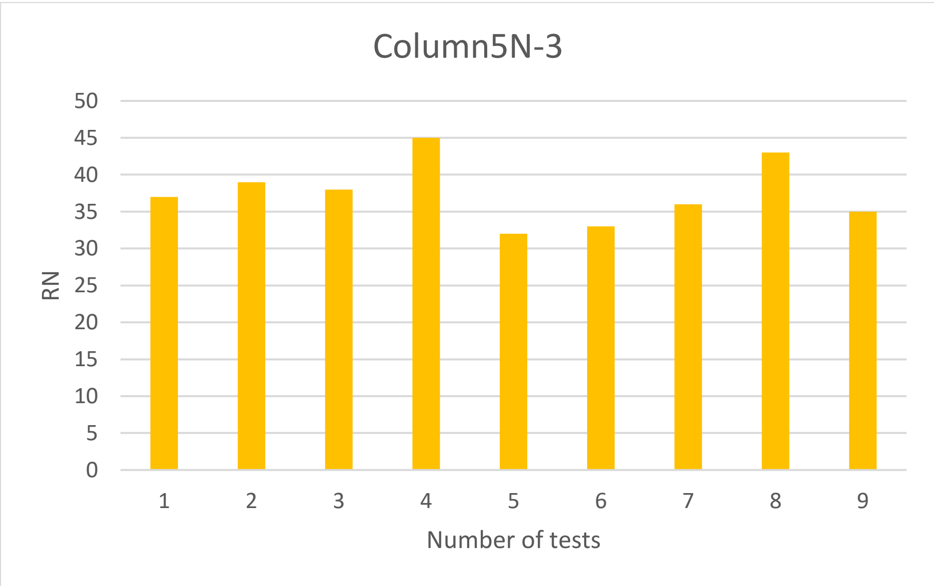




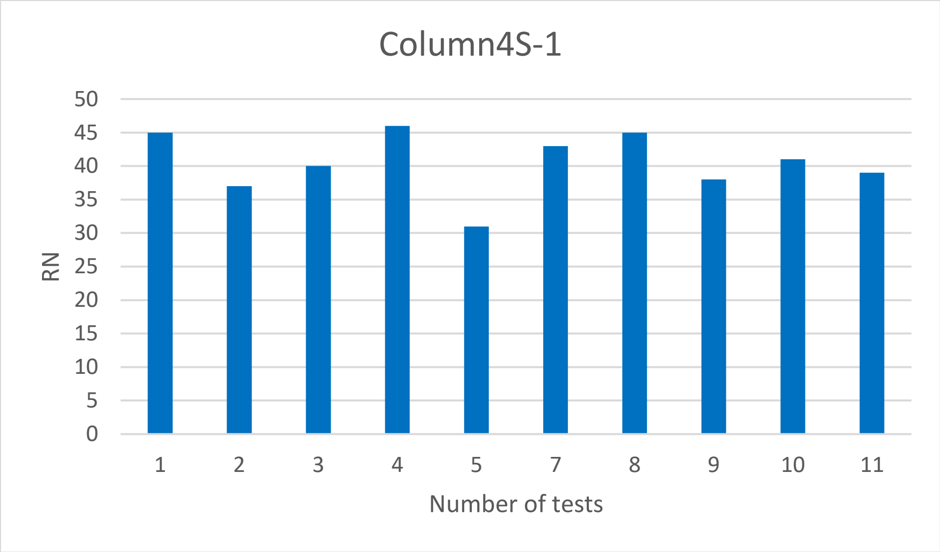
Column5N-1
RN
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
Number of tests

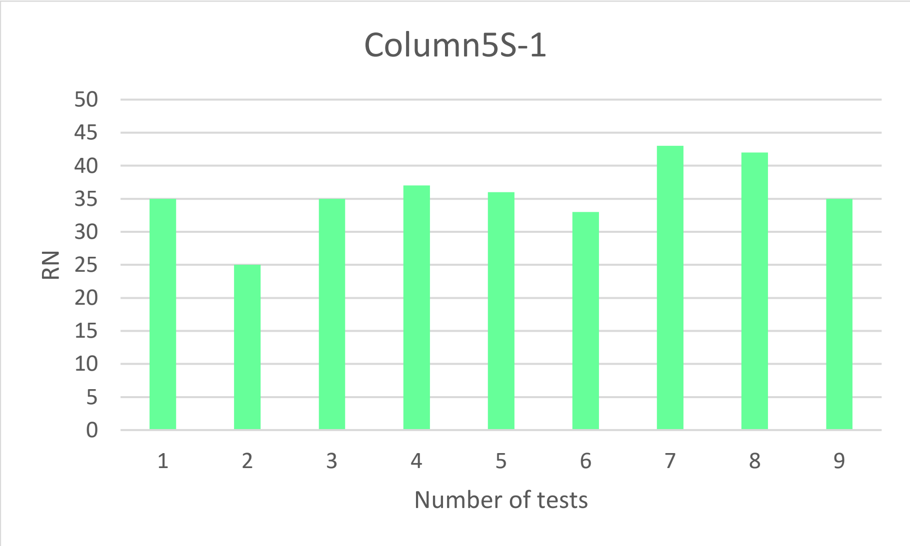


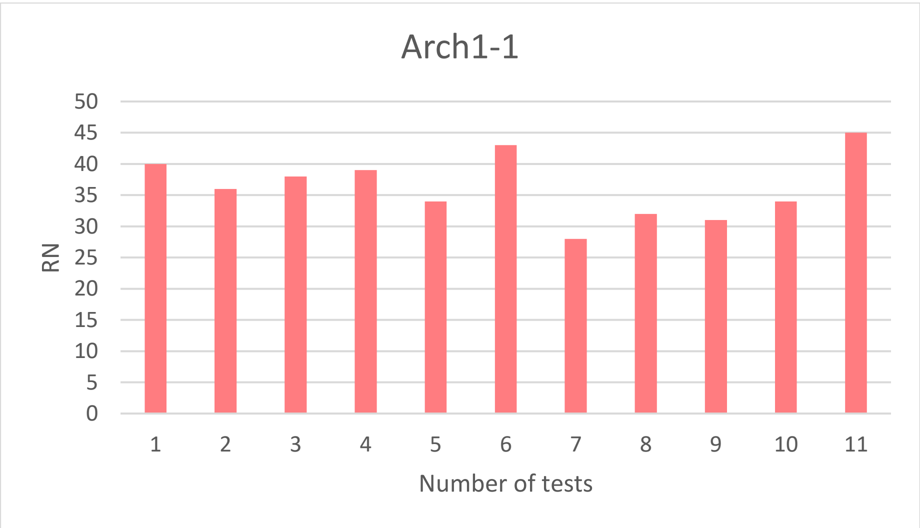
Column5N-2
RN
60
50
40
30
20
10
0
1
3
4
5
6
7
8
9
Number of tests

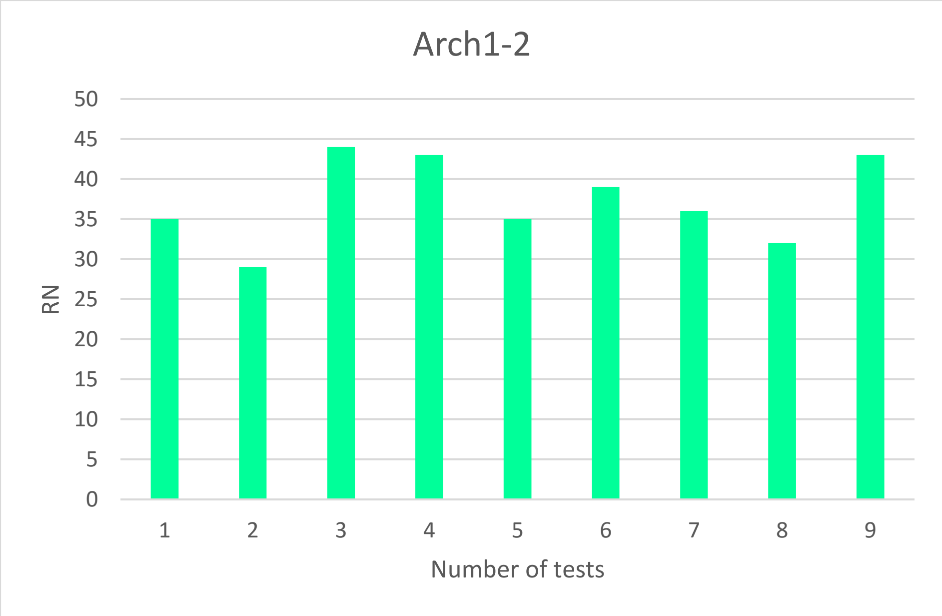


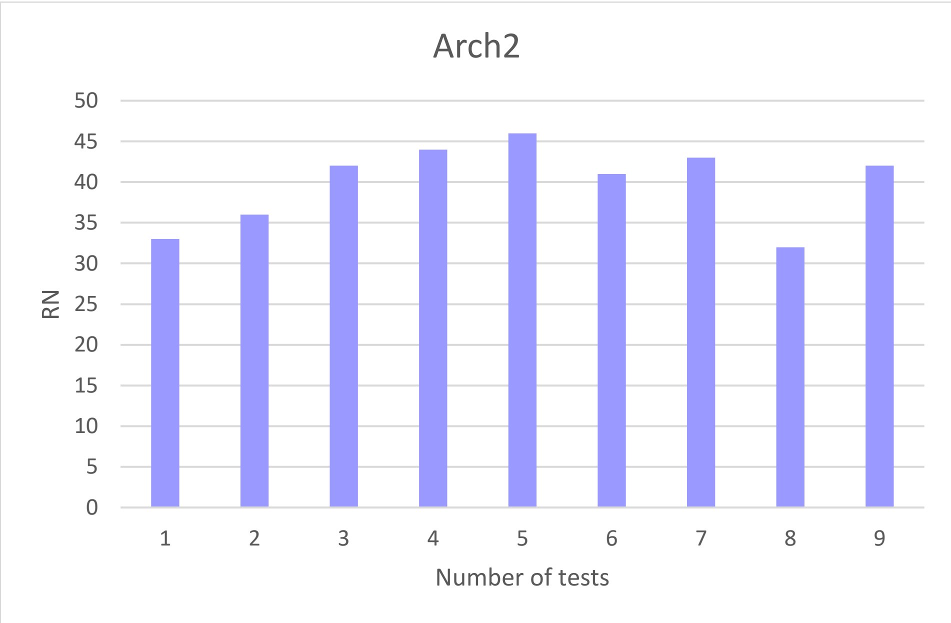
Column5N-3
RN
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
Number of tests

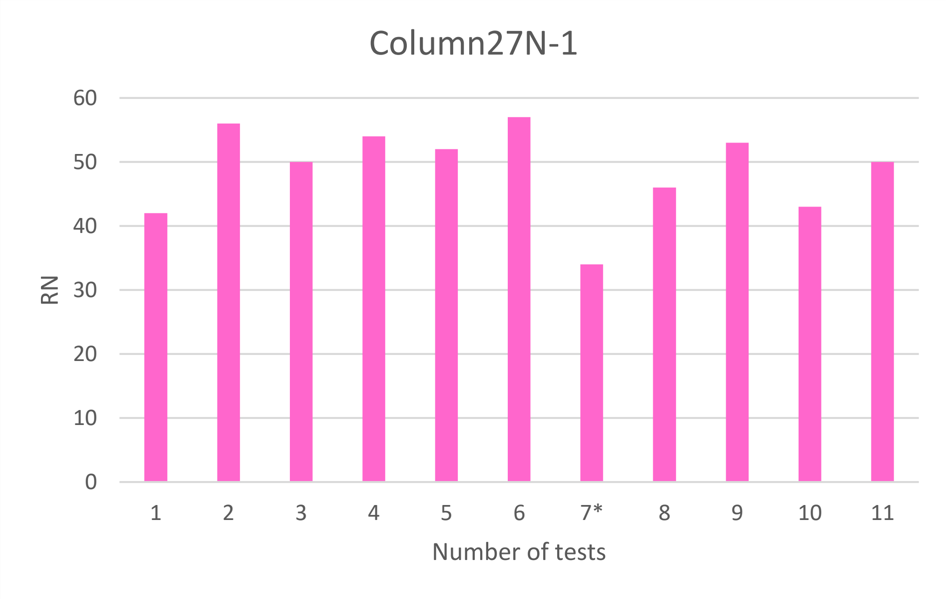

Column4S-1
RN
Number of tests
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
7
8
9
10
11

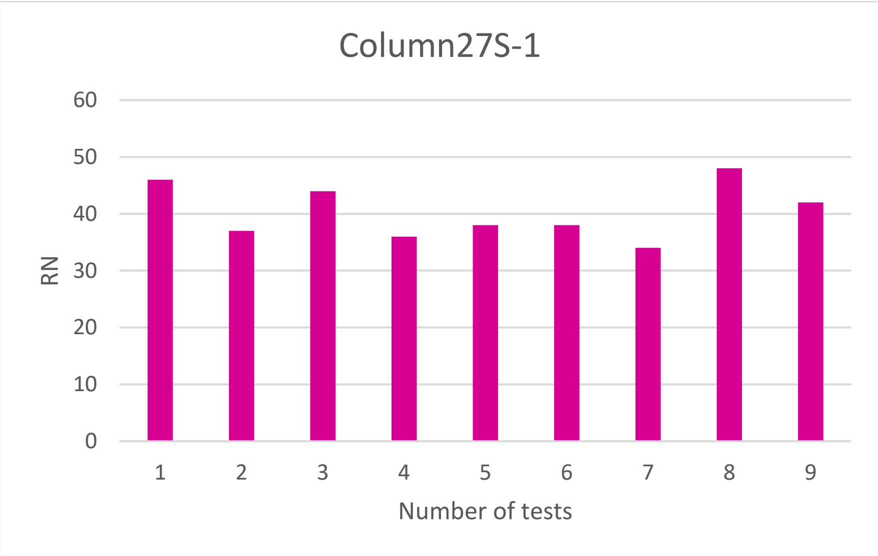

Column5S-1
RN
Number of tests
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9


Arch1-1
RN
Number of tests
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
10
11


Arch1-2
RN
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
Number of tests


Arch2
RN
50
45
40
35
30
25
20
15
10
5
0
1
2
3
4
5
6
7
8
9
Number of tests





8.4 Outlier tests

8.4.1 Schmidt hammer outlier test

Test location	4N1	4N2	5N1	5N2	5N3	4S1	5S1	Arch1-1	Arch1-2	Arch2	27N	27S
#	1	2	1	2	3	1	1	1	2	1	1	1
1	31	41	41	45	37	45	35	40	35	33	42	46
2	40	41	38	35	39	37	25	36	29	36	56	37
3	37	44	43	48	38	40	35	38	44	42	50	44
4	40	45	31	37	45	46	37	39	43	44	54	36
5	42	43	37	38	32	31	36	34	35	46	52	38
6	41	40	31	28	33	43	33	43	39	41	57	38
7	43	41	31	27	36	45	43	28	36	43	34	34
8	35	43	37	35	43	38	42	32	32	32	46	48
9	37	28	27	51	35	41	35	31	43	42	53	42
						39		34			43	

						26		45			50	
Median	40	41	37	37	37	40	35	36	36	42	50	38
30% median	12	12	11	11	11	12	11	10.8	10.8	12.6	15	11.4
Lower limit	28	29	26	26	26	28	25	25.2	25.2	29.4	35	26.6
Upper limit	52	53	48	48	48	52	46	46.8	46.8	54.6	65	49.4
Set Valid	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

8.4.2 UPV test results Grubb test

#	Test location	UPV result
1	Column 4 N-1	4076
2	Column 4 N-2	4033
3	Column 4 N-3	3999
4	Column 4 N-4	3968
5	Column 5 N-1	4507
6	Column 5 N-2	4376
7	Column 5 N-3	4192
8	Column 5 N-4	4247
9	Arch 1-1	3867
10	Arch 1-2	2674
11	Arch 1-3	3097
12	Column 27 N-1	4466
13	Column 27 N-2	4601
14	Column 27 N-3	4576
15	Column 27 N-4	4501
16	Column 27 S-1	4810
17	Column 27 S-2	4740
18	Column 27 S-3	4715
19	Column 27 S-4	4725
20	Arch 2-1	3969
21	Arch 2-2	4305
22	Arch 2-3	4044
23	Arch 2-4	4220
	Mean	4204.7
	Sample SD	508.0
	Lowest	2674.0
	Highest	4810.0
	Mean-Lowest	1530.7
	Highest-Mean	605.3

According to EN 13791:2019, the Grubb test is applied to the value with maximum absolute difference with the value of the mean.

Hence, the Grubb test is applied to the lowest value

$$\frac{f_{\mathrm{c,is,lowest}} - f_{\mathrm{c,m(n)}}}{s} = \left|\frac{2674 - 4204}{508}\right| = 3.013 < G_{\mathrm{p}} = 3.08$$

Hence, it is not outlier

8.4.3 Core results Grubb test

#	Test location	Cores
1	4N	28.1
2	4N	47.1
3	4N	37.4
4	5N	40.2
5	Arch1	45.8
6	Arch2	33.4
7	27NN	33.7
8	27NN	43.4
9	27NN	38.3
	Mean	38.6
	Sample SD	6.2502
	Lowest	28.1
	Highest	47.1
	Mean-Lowest	10.5
	Highest-Mean	8.5

According to EN 13791:2019, the Grubb test is applied to the value with maximum absolute difference with the value of the mean.

Hence, the Grubb test is applied to the lowest value

$$\frac{f_{\mathrm{c,is,lowest}} - f_{\mathrm{c,m(n)}}}{s} = \left|\frac{28.1 - 38.6}{6.25}\right| = 1.68 < G_{\mathrm{p}} = 2.38$$

Hence, it is not outlier

8.5 Matlab codes used for creating combinations

```
Combination of samples of 3 cores
clear, clc, close all;
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=3;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3)};
filename = 'comb3.xlsx';
writecell(C,filename,'Sheet','comb3','Range','B2')

Combination of samples of 4 cores
clear, clc, close all;
```

```
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=4;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3);C(:,4)};
filename = 'comb4.xlsx';
writecell(C,filename,'Sheet','comb4','Range','B2')
```

Combination of samples of 5 cores

```
clear, clc, close all;
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=5;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3);C(:,4);C(:,5)};
filename = 'comb5.xlsx';
writecell(C,filename,'Sheet','comb5','Range','B2')
```

Combination of samples of 6 cores

```
clear, clc, close all;
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=6;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3);C(:,4);C(:,5);C(:,6)};
filename = 'comb6.xlsx';
writecell(C,filename,'Sheet','comb6','Range','B2')
```

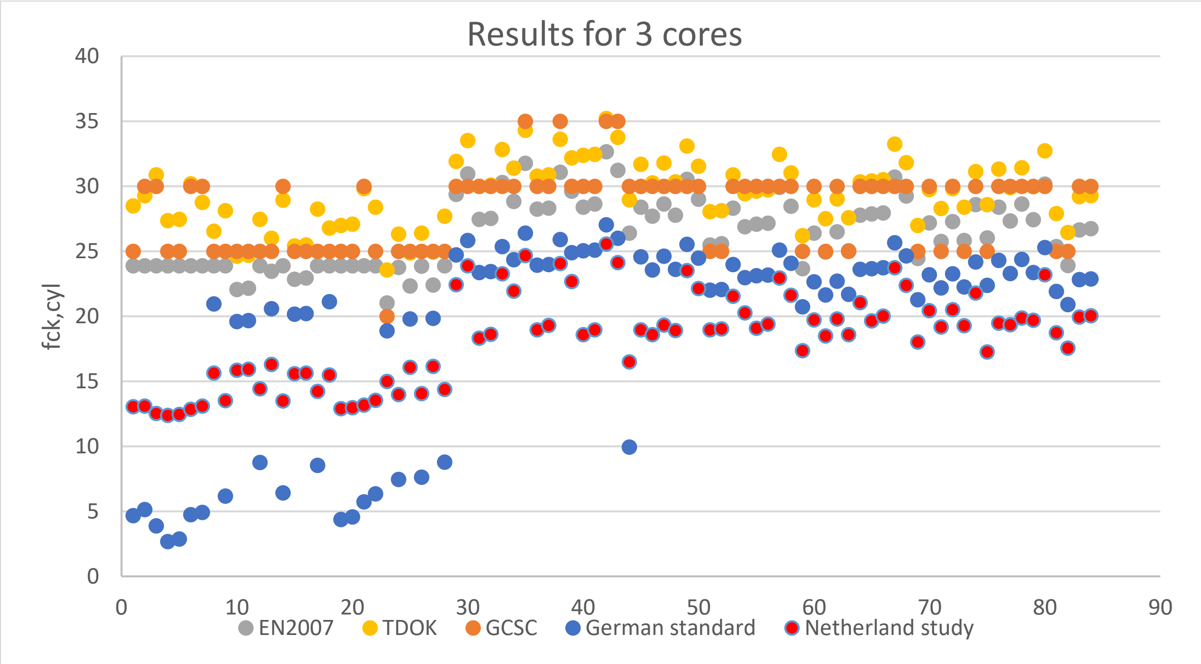
Combination of samples of 7 cores

```
clear, clc, close all;
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=7;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3);C(:,4);C(:,5);C(:,6);C(:,7);};
filename = 'comb7.xlsx';
writecell(C,filename,'Sheet','comb7','Range','B2')
```

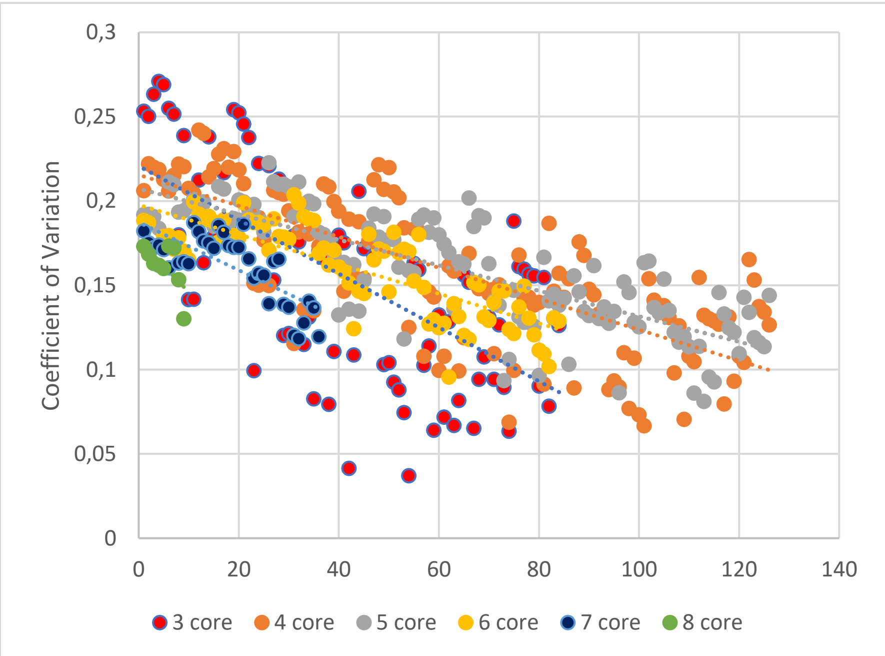
Combination of samples of 8 cores

```
clear, clc, close all;
v=['A';'B';'C';'D';'E';'F';'G';'H';'I'];
k=8;
C = nchoosek(v,k);
display (C);
C = { C(:,1); C(:,2);C(:,3);C(:,4);C(:,5);C(:,6);C(:,7);C(:,8)};
filename = 'comb8.xlsx';
writecell(C,filename,'Sheet','comb8','Range','B2')
```

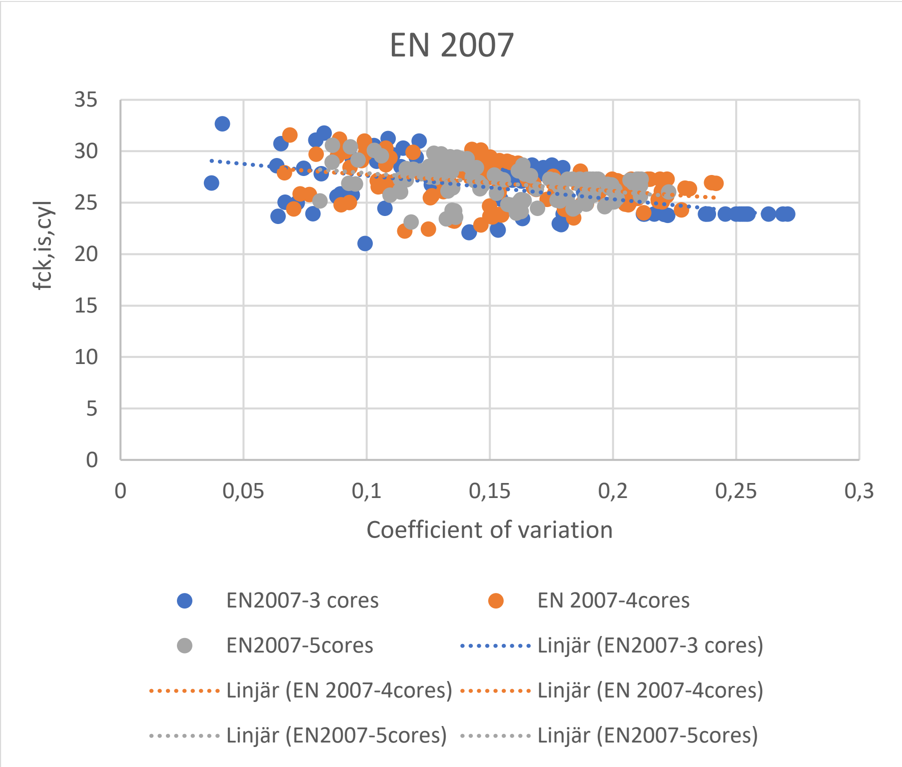
8.6 Results of estimation of in-situ compressive strength for 3 cores of different methods

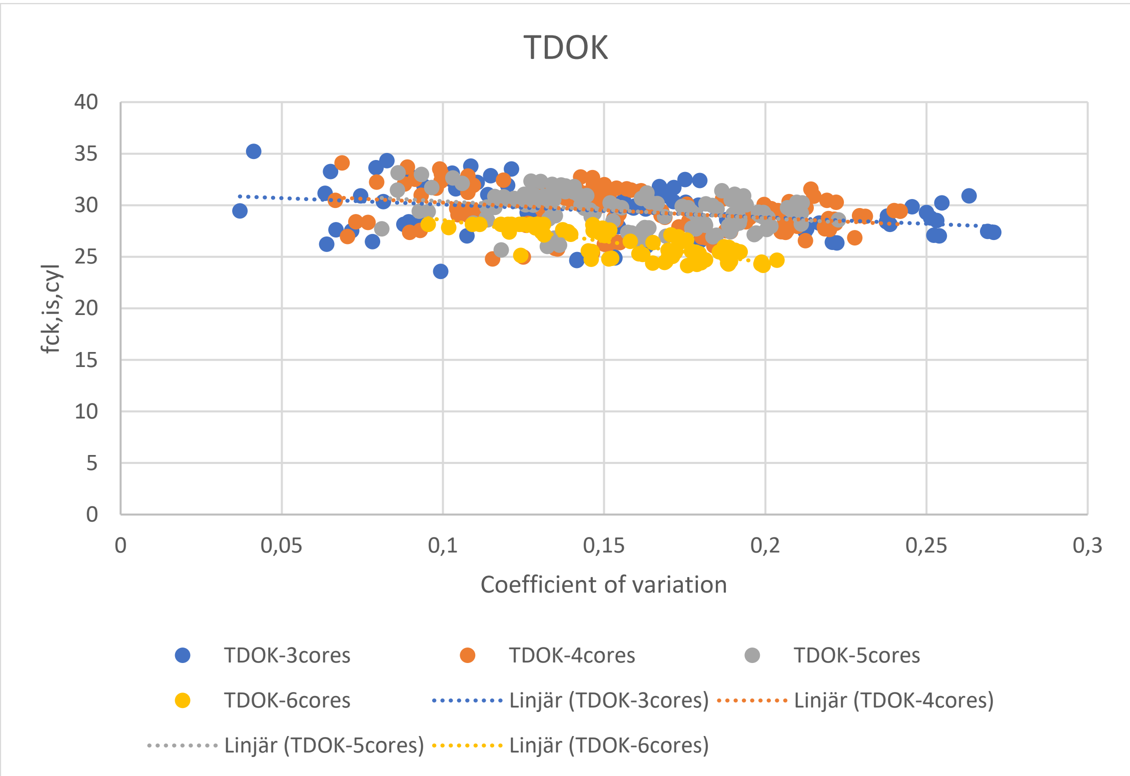


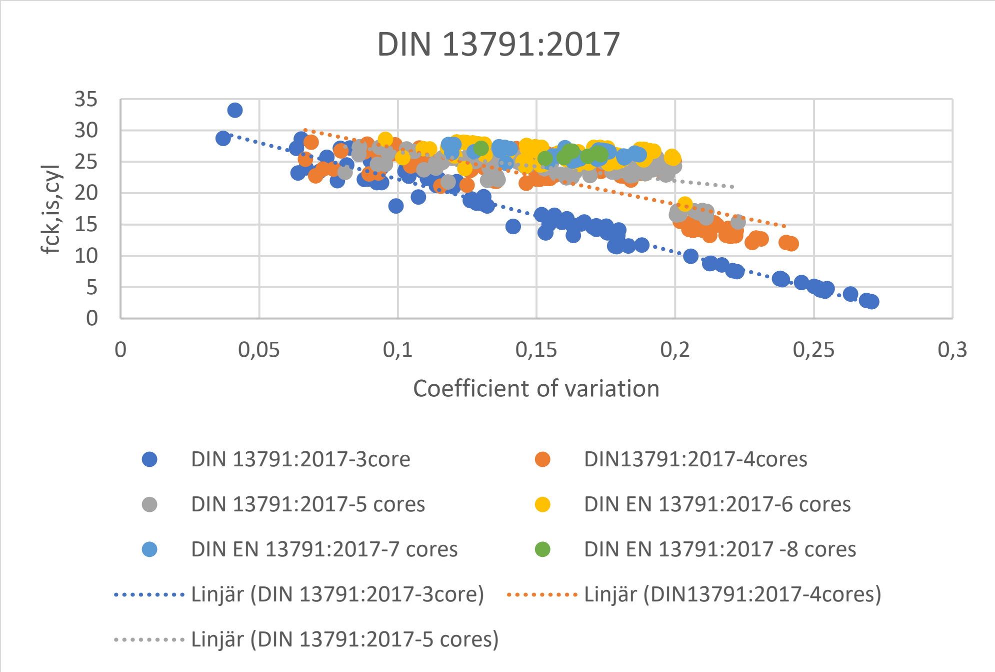
8.7 Relation between coefficient of variation with number of cores

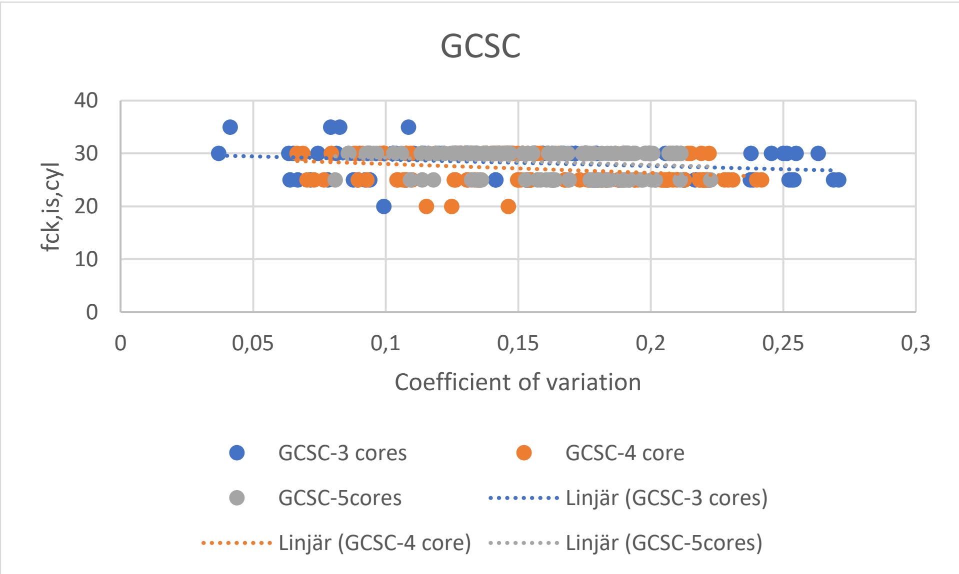


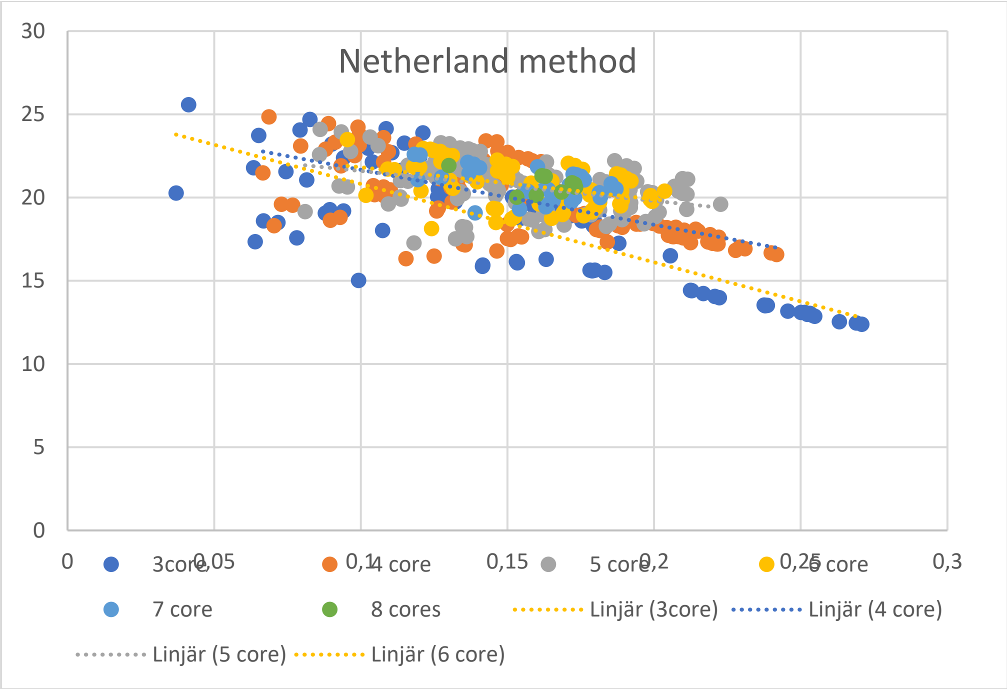
8.8 Relation between coefficient of variation with number of cores for different methods






DIN 13791:2017
fck,is,cyl
Coefficient of variation
DIN 13791:2017-3core
DIN13791:2017-4cores
DIN 13791:2017-5 cores
DIN EN 13791:2017-6 cores
DIN EN 13791:2017-7 cores
DIN EN 13791:2017 -8 cores
Linjär (DIN 13791:2017-3core)
Linjär (DIN13791:2017-4cores)
Linjär (DIN 13791:2017-5 cores)

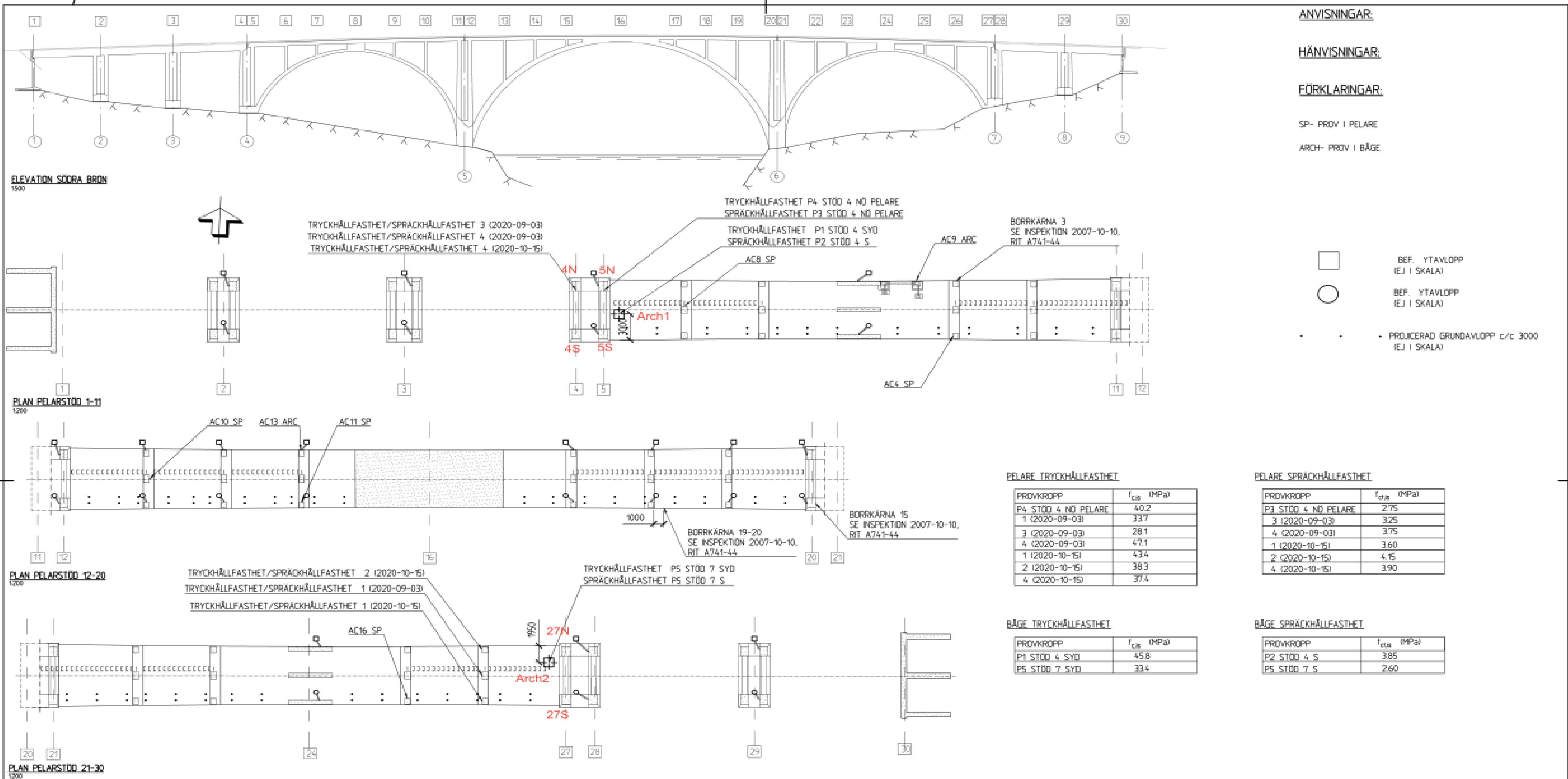

GCSC
fck,is,cyl
Coefficient of variation
GCSC-3 cores
GCSC-4 core
GCSC-5cores
Linjär (GCSC-3 cores)
Linjär (GCSC-4 core)
Linjär (GCSC-5cores)



Netherland method
30
25
20
15
10
5
0
0
0,05
0,1
0,15
0,2
0,25
0,3
3core
4 core
5 core
6 core
7 core
8 cores
Linjär (3core)
Linjär (4 core)
Linjär (5 core)
Linjär (6 core)

8.9 Test locations on Skuru bridge

BILAGA 1



PELARE TRYCKHÅLLFASTHET

PROVKROPP	f_{cis} (MPa)
P4 STÖD 4 NÖ PELARE	40.2
1 (2020-09-03)	33.7
3 (2020-09-03)	28.1
4 (2020-09-03)	47.1
1 (2020-10-15)	43.4
2 (2020-10-15)	38.3
4 (2020-10-15)	37.4

PELARE SPRÄCKHÅLLFASTHET

PROVKROPP	$f_{ct,is}$ (MPa)
P3 STÖD 4 NÖ PELARE	2.75
3 (2020-09-03)	3.25
4 (2020-09-03)	3.75
1 (2020-10-15)	3.60
2 (2020-10-15)	4.15
4 (2020-10-15)	3.90

BÅGE TRYCKHÅLLFASTHET

PROVKROPP	f_{cis} (MPa)
P1 STÖD 4 SYD	45.8
P5 STÖD 7 SYD	33.4

BÅGE SPRÄCKHÅLLFASTHET

PROVKROPP	$f_{ct,is}$ (MPa)
P2 STÖD 4 S	3.85
P5 STÖD 7 S	2.60

PELARE KLORIDHALT %

PROVKROPP	MÄTT TB (mm)	0-15mm	15-45mm	45-75mm	HÖJD
1 (2020-07-07)	30	0.133	0.231	0.180	1.2m FRÅN BASEN
AC4 SP	30	0.415	0.609	0.350	VID BASEN
AC8 SP	30	0.311	0.193	0.035	VID BASEN
AC10 SP	30	0.106	0.140	0.044	1.5m FRÅN BASEN
AC11 SP	30	1.258	1.107	0.398	VID BASEN
AC16 SP	30	0.415	0.514	0.101	1.5m FRÅN BASEN

BÅGE KLORIDHALT %

PROVKROPP	MÄTT TB (mm)	0-15mm	15-45mm	45-75mm	LÄGE
AC9 ARC	30	0.398	0.200	0.030	NORRA SIDAN
AC13 ARC	30	1.258	1.107	0.398	NORRA SIDAN

PELARE KLORIDHALT % ENLIGT INSPEKTION 2007-10-10

PROVKROPP	MÄTT TB (mm)	0-10mm	10-30mm	30-60mm	60-100mm	LÄGE
BORRKÄRNA 3	30	0.500	0.330	0.090	0.030	0.55m FRÅN TOPPEN
BORRKÄRNA 15	30	0.300	0.700	-	-	6.5m FRÅN TOPPEN

BÅGE KLORIDHALT % ENLIGT INSPEKTION 2007-10-10

PROVKROPP	MÄTT TB (mm)	0-10mm	10-30mm	30-60mm	60-100mm	LÄGE
BORRKÄRNA 19	30	0.310	0.450	0.040	-	SÖDRA SIDAN
BORRKÄRNA 20	30	0.760	1.010	-	-	SÖDRA SIDAN

GAMLA SKURUBROANA
PROVTAGNINGSRAPPORT

A124377-4-02-PPA-002
BILAGA 1

PROVTAGNINGSLÄGE OCH UPPMÄTA DATA
PELARE OCH BÅGE
2-7-1 (SÖDRA BRON)

COWI

FOSO

MMAH FOSO

STOCKHOLM 2020-10-21

1:500 , 1:200

B1-03

TRITA-ABE-MBT-2118