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Plastic Concrete for Cut-Off Walls

by

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Abstract

Plastic Concrete plays a key role in the remediation of earthen dams using cut-off walls to counter dam seepage. However, Plastic Concrete has yet to be thoroughly understood, since little attention has been given to this material in literature. The principal objective of this report is to set out the fundamental material science parameters, which describe Plastic Concrete's mechanical and hydraulic behaviour as well as describing the mix design and application of Plastic Concrete. For this, an extensive and comprehensive literature review was carried out. The results show that Plastic Concrete can hereby be considered to be a low-strength, low-stiffness impervious concrete with a high deformation capacity under load and the capability of sustaining larger strains than normal concrete. This study further identifies reference values, which may be used in cut-off wall design. All in all, the research results represent a further step towards the understanding of Plastic Concrete material behaviour.

Table of Contents

1. Introduction

1.1 Background

The worldwide aging infrastructure is a reason for concern in many countries. Unfortunately, only when a catastrophic failure of some infrastructure occurs, this topic obtains public awareness. A key example for the systematic, catastrophic failure of embankment dams and levees occurred in 2005 during the Katrina and Rita Hurricanes in the North-American Gulf Shore area [33]. Most recently, in February 2017, the Oroville Dam Failure further emphasized the critical situation of many dam infrastructures [75]. These catstrohpes have highlighted the need for remediation works on a worldwide scale. In the United States alone, approximately 91,000 dams are currently in need of some type of repair in varying degrees of deterioration [33, p.1] [126]. Furthermore, approximately 86% of these dams are earthen dams, where the average age of these dams is currently 50 years [126]. Various failure modes are possible for earthen dams, ranging from dam over-topping and inadequate maintenance to foundation defects and slope instability. The latter generally occurs through water seepage below the dam body causing a reduction in internal friction and causing the dam to slip. Moreover, seepage may cause piping within the dam and even a hydraulic heave failure to occur. Therefore, major concern is raised regarding dam safety and various dam repair and remediation programs have been initiated worldwide.

A common solution to counter dam seepage is the design and construction of cut-off walls [74]. This may occur in the design phase of new earthen dams or be constructed during remediation of an existing dam. Figure 1.1 shows a possible design concept.

The planned cut-off wall is hereby extended into an underlying impervious stratum, e.g. rock [125, p.46]. For the cut-off wall construction and material choice various possibilities exist. Cut-off walls may be constructed with mixed-in-place technologies, grouting methods or through excavated and backfilled cut-off constructions [33]. Each of these construction methods has its advantages and disadvantages, for which the appropriate choice of method greatly depends on,

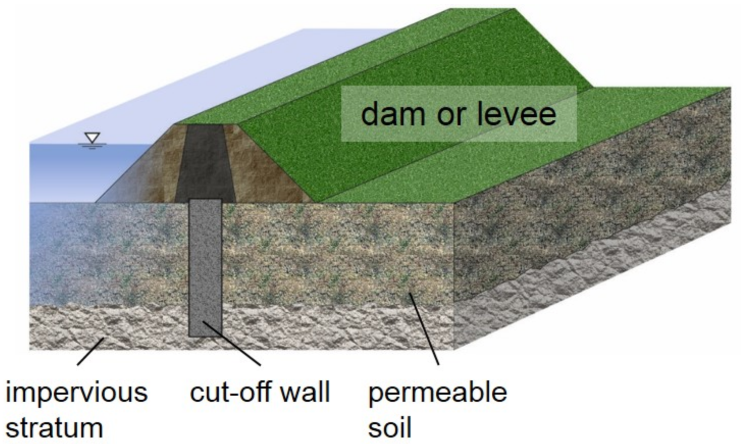


Figure 1.1: Schematic of an Earth Dam with Cut-Off Wall

amongst others, time, cost, geographical location, technological and geological factors [26, 33]. The most effective cut-off walls for seepage control can be constructed with excavated slurry-trench walls, especially for greater depths [74,133]. For these, a wide range of backfill materials may be used depending on specific project requirements. For this reason, excavated cut-off walls are of preferred choice for modern dam remediation. As backfill materials a wide range of possibilities exist, e.g. standard concrete, soil-cement, soil-cement-bentonite, cement-bentonite or Plastic Concrete [85, p.VIII-2] [125, p.16-7].

1.2 Definition & Field of Application

Generally Plastic Concrete can be considered to be a low-strength, low-stiffness impervious concrete with a high deformation capacity under load. The European standard EN 1538 [47] defines Plastic Concrete as a low-strength, low Young's modulus concrete capable of sustaining larger strains than normal concrete. This material should have high deformability and low permeability while ensuring sufficient material workability and strength [47, p.19]. Similarly, the DWA M512-1 guideline [56] considers Plastic Concrete to be a material which has a low hydraulic conductivity and high deformability. This guideline also gives reference values and recommendations for cut-off walls in general. The United States Bureau of Reclamation's Design Standard No. 13 [125] states that Plastic Concrete is a regular concrete which, through the addition of bentonite, becomes less stiff and can therefore undergo greater strains before cracking compared to usual concrete walls [125, p.16-47].

Plastic Concrete is furthermore placed under a supporting fluid (e.g. bentonite suspension) using the so-called tremie method. With this, Plastic Concrete has less potential for construction defects than the conventional soil-bentonite or cement-bentonite backfilling [72]. In addition, the high strain capacity of Plastic Concrete is of great advantage when ductile walls are needed if unequal deformations of the cut-off wall, large annual reservoir fluctuations or significant seismic events are expected, causing significant bending strains to be induced [33] [35, p.570] [125, p.58]. Through the highly ductile behaviour of the material, the rupture probability can be decreased and wide, open cracks can be reduced to a minimum hindering a material permeability increase [33, p.230] [77]. In specific situations Plastic Concrete can also be used for the containment of contaminated soils and other applications [27, 130, 136]. Plastic Concrete has therefore been widely used in dam remediation for many years, with projects like the Sylvenstein Dam (Germany) [95], Hinze Dam (Australia) [24] or Bagatalle Dam (Mauritius) [25].

1.3 Cut-Off Wall Construction

As mentioned previously, the main task of a cut-off wall is to mitigate seepage, generally below an existing or new dam. With the ever evolving technology in specialist foundation engineering depths in excess of 120 m and ground strengths greater than 160 MPa can be safely excavated [31, p.305]. The most common method is the excavation of a slurry-trench wall with the use of clamshells or cable-suspended hydro-cutters, as shown in Figure 1.2, ①.

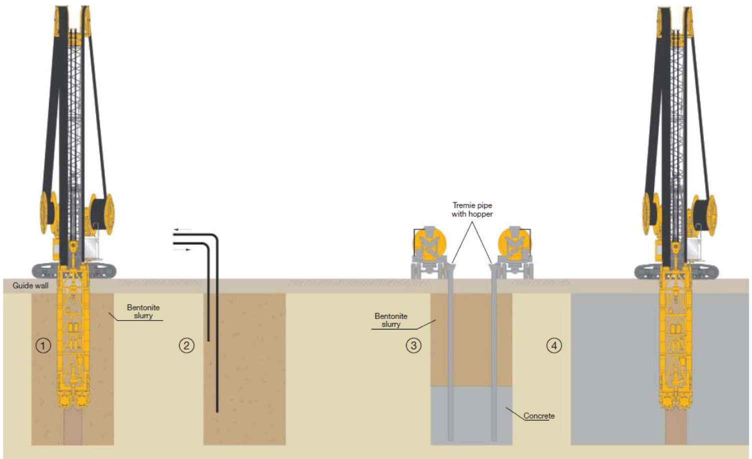


Figure 1.2: Construction of a Cut-Off Wall with the Two-Phase Method [27]

The excavating method performance and choice greatly depends on depth, ground strength and boulder content [31, p.308] [124]. For example only the cable-suspended hydro-cutters are capable of greater depths and can be maintained under tight tolerances [68] [125, p.46]. During excavation the panels are continuously filled with a bentonite or polymer slurry to hold the trench in place.

This working slurry is then cleaned (de-sanded) or replaced by a fresh slurry to ensure homogeneous conditions for tremie concreting [66] as shown in Figure 1.2, ②. This slurry is then replaced with Plastic Concrete using the tremie placement method, whereby concrete is fed through hoppers into metal pipes replacing the slurry from bottom to top [125, p.46], as shown in Figure 1.2, ③. This also clearly differentiates Plastic Concrete cut-off walls from cement-bentonite walls since the latter are produced in a single phase with self-hardening slurries without the use of the tremie method [55, 124]. Only through the use of the two-phase method, panels with greater depth are even possible due to the long excavation times and increases suspension of soil [86, p.N28].

Concrete placement using the tremie method must however be done with great care since various influencing factors exist [70]. The bentonite cake may for example remain in the construction joints and therefore adequate construction expertise is essential for the correct construction of cut-off walls [31, p.309]. The excavated panels are generally between 2.50 m and 7.50 m in length [124, p.37]. The panels width ranges between 0.6 m for low depths and 1.2 m for greater depths. The added width is generally required to assist in maintaining overlap between adjacent panels and ensure imperviousness of the cut-off wall [74]. To construct the complete cut-off wall, construction is planned in an alternating sequence of primary and secondary panels, as shown in Figure 1.2, ④.

Although the slurry-trench method is the most widespread excavating method, for small depth cut-off walls Plastic Concrete has also been placed with the secant pile method. For example, the 30 m deep cut-off wall at the Papadia Dam (Greece) was constructed using this method, whereby a minimum pile overlap of 0.7 m was used to ensure cut-off wall imperviousness [3, p.1107].

1.4 Requirements

To ensure the imperviousness of Plastic Concrete cut-off walls, these walls must meet various requirements. Excluding cost, anticipated hydraulic gradient across the cut-off wall and depth of cut-off wall are two of the main consideration influences [125, p.8]. The mix design of cut-off wall materials is hardly standardised, since regulatory authorities are less concerned with the nature of the material, than with its performance [86, N13]. Plastic Concrete however, has many more configuration possibilities than cement-bentonite, which is a distinct class of material with a distinct range of typical properties [86, N13]. Most commonly, standards and guidelines limit the compressive strength, Young's modulus and hydraulic conductivity of Plastic Concrete.

Compressive Strength Requirements

For the required Plastic Concrete compressive strength various standards and guidelines give varying requirements. ICOLD Bulletin No. 51 recommends the use of the lowest compressive strength possible, to obtain a material having the highest possible deformability [87, p.11]. Similarly, the DWA M512-1 guideline requires a minimal unconfined compressive strength of 0.3 MPa at 28 days to account for sufficient erosion resistance [56]. The compressive strength requirements do not necessarily have to be at 28 days, since Plastic Concrete strength is known to increase strongly over time (as will be described in subsection 3.2.1) and is not required for construction purposes. For this reason DIN EN 1538 suggests, that long term strength and deformability may be accounted for material design and testing ages [47, p.19]. In line with this, the Austrian standard ÖNORM B4452 requires Plastic Concrete to achieve an unconfined compressive strength (UCS) of at least 0.5 MPa at 90 days, or alternatively an UCS of 0.3 MPa at 7 days in the event of a water table draw-down within the first 90 days [108, p.10]. It should also be noted that Plastic Concrete strength is generally higher than that of cement-bentonite mixtures where a minimum compressive strength of 0.1 MPa at 28 days is requested by some guidelines [86]. Cement-bentonite projects have also been known to achieve a compressive strength of 0.5 to 1.5 MPa [55, p.39].

In practical Plastic Concrete applications the compressive strength has been required to range between 1.0 and 2.0 MPa. Most recently, the Bagatelle Dam Plastic Concrete cut-off wall required a compressive strength of 1.0 to 1.5 MPa at 28 days [25, p.39]. The Hinze Dam Plastic Concrete cut-off wall was expected to achieve compressive strengths of 2.0 to 4.0 MPa at 28 days [24] [31, p.310]. It should however always be taken into account, that due to the interdependance between concrete compressive strength and elastic modulus (which will be adressed further on in this report), the required compressive strength should be considered a target strength and not a minimal strength [95].

Deformation Requirements

ICOLD Bulletin No. 51 recommends the elastic modulus of Plastic Concrete to be four to five times greater than that of the surrounding soil [87, p.11]. This aims to achieve a material presenting similar deformation characteristics to the surrounding soil [87, p.11], thus decreasing the relative settlement between cut-off wall and surrounding soils hereby reducing the so called arching effect [100]. This in turn reduces the stress applied on cut-off wall material [77]. The Bagatelle Dam project required for example a deformation modulus (with geotechnical testing standards) of 100 to 150 MPa at 28 days [25, p.39]. The testing conditions for the deformation modulus (e.g. testing age, strain level, drainage conditions, confining pressure & strain rate) are however not often standardised [86, p.N15]. Some guidelines also require a specific strain at failure value to be achieved [86, p.N14].

Hydraulic Conductivity Requirements

Regarding the hydraulic conductivity of Plastic Concrete, most standards and guidelines require similar values to be achieved. The DWA M512-1 guideline states that with a hydraulic conductivity $k_3 \leq 1 \cdot 10^{-8}$ m/s, a hydraulic gradient of $i = 100$ may be safely absorbed by a Plastic Concrete cut-off wall. For cement-bentonite cut-off walls the British Institution of Civil Engineers recommends a target permeability of $k = 1 \cdot 10^{-9}$ m/s [86, p.S5]. Interestingly however, due to the inherent variability within the material this guideline suggests that 80 % of specimens have a permeability of less than $1 \cdot 10^{-9}$ m/s, 95% of less than $1 \cdot 10^{-8}$ m/s and no single values less than $5 \cdot 10^{-8}$ m/s, when testing at an age of 90 days or later [86, p.S5].

In practical applications the permeability requirements have more recently moved from $1 \cdot 10^{-8}$ m/s to $1 \cdot 10^{-9}$ m/s [86, p.N13]. The permeability of the Hinze Dam (Australia) was required to be $k \leq 1 \cdot 10^{-9}$ m/s [31, p.310] [24, p.47] and in the Sylvensteindamm (Germany) a permeability $k \leq 1 \cdot 10^{-9}$ m/s was also expected [95]. Since permeability is more sensitive to testing age and time under permeation, with tighter specification requirements having been established, testing has generally been recommended to move to at least 90 days [86, p.N13]. For example, the Austrian standard OENORM B 4452 requires Plastic Concrete permeability testing to be performed up to 90 days of age [108, p.9].

1.5 Problem Defintion

Current design of Plastic Concrete is however simplistic with a linear-elastic material model being used for material modelling. For cut-off wall and dam safety it is therefore of utmost importance that Plastic Concrete is realistically designed and sufficiently understood [29, 30]. Most importantly neither the time-dependant properties (e.g. creep behaviour) nor the deformation properties (e.g. high ductility) are considered when designing cut-off wall materials [30]. To date, there are major uncertainties in the design and construction of all types of cut-off walls, which must be addressed [125, p.8].

The following report therefore aims to review the State-of-the-Art on Plastic Concrete for cut-off walls and recommend an approach for the correct modelling of Plastic Concrete behaviour. In chapter 2 the raw materials, the mixture composition and the mixing sequence for Plastic Concrete are set out. In this chapter the fresh properties of Plastic Concrete are also highlighted. In chapter 3 the mechanical behaviour of hardened concrete is described in detail. Hereby the different material testing standards for Plastic Concrete strength, elastic modulus and creep behaviour are explained against the available literature. In chapter 4 the hydraulic behaviour of Plastic Concrete is described. Finally, in chapter 5 the main results of Plastic Concrete behaviour are briefly summarised and the needs for future research are proposed.

2. Mix Design

Contemporary standard concrete is considered a five-phase construction material composed of cement, water, aggregate (sand and gravel), additions (e.g. supplementary cementitious materials) and admixtures (e.g. set-retardants, superplasticisers, stabilisers). Plastic Concrete can also be considered a five-phase construction material, however in differing proportions to those usually mixed with standard concretes and containing bentonite as an additional constituent. In section 2.1 the various materials used in the mix design of Plastic Concrete are mentioned and their specific properties and purposes are detailed. Following on, in section 2.2 the mixture composition of Plastic Concrete is described and placed within concrete technology context. In addition, in section 2.3 the existing mixing sequence possibilities are described and weighed out against one another. Finally, in section 2.4 the testing requirements of Plastic Concrete fresh properties are given.

2.1 Materials

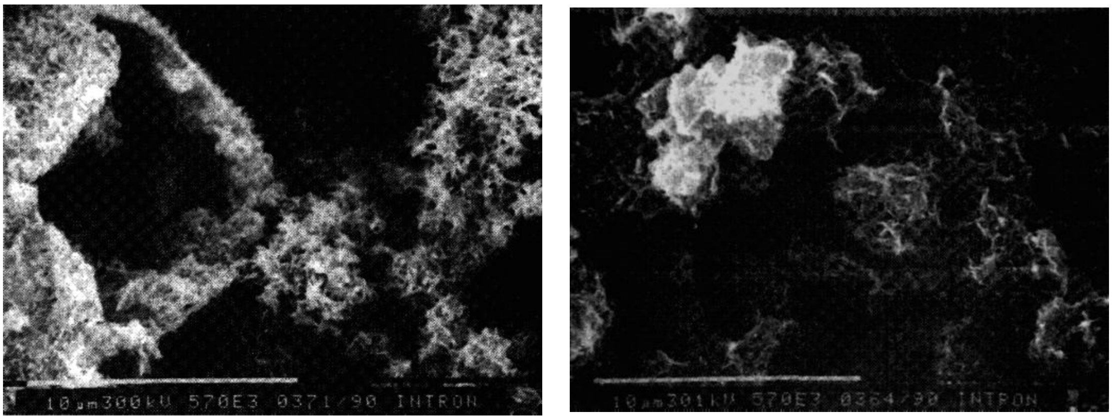
As mentioned previously, Plastic Concrete can be considered a five-phase construction material. Here cement, water, aggregate (mainly sand and fine gravel) are used in combination with bentonite as an additive and sometimes admixtures to obtain a highly ductile and impermeable material. However, the composition of Plastic Concrete is not limited to the aforementioned components and could be produced using other supplementary cementitious materials (e.g. fly ash). In the following sections the most often used materials are described.

2.1.1 Cement

Cement is a type of binder with adhesive and cohesive properties commonly used in the construction industry capable of binding building materials together. Hydraulic cements are most commonly produced by mixing calcareous and argillaceous materials together and burning them at high temperatures (i.e. clinkering temperature) and grinding the resulting clinker into a powder [105, p.2]. In the presence of water this dry powder undergoes a chemical reaction becoming adhesive and forming calcium silicate hydrate (C-S-H) crystals which consolidate and strengthen the structure. The variety of hydraulic cements available on the market is manifold, as the use of cement is widespread in the construction industry with many applications (e.g. ordinary Portland cement, sulfate-resisting cement, blastfurnace cement). Their dry density therefore also range from 2.85 g/cm^3 to 3.5 g/cm^3. In Europe the varying types of cement are regulated within a single CEN-standard, which in Germany corresponds to DIN EN 197-1 [48]. Most notably, the European standard classifies the various cements by composition, whereby these are divided in five categories from CEM I (Portland cement) to CEM V (composite cement). In the Unites States various standard apply to describe the varying type of cements. Firstly ASTM C150/C150M [7] regulates ordinary Portland cement, while ASTM C595/595M [11] standardises blended hydraulic cements. Some physical properties are also regulated separately in the performance specification ASTM C1157/C1157M [6]. This should especially be taken into account since an ASTM C150/C150M [7] Type I cement corresponds to a DIN EN 197-1 [48] CEM I class cement. However, a DIN EN 197-1 [48] CEM III/A class cement does not correspond to a ASTM C150/C150M [7] Type III cement, but instead a Type I (36<S<65) cement. For more information regarding cement classification and terminology, refer to [8, 105].

For Plastic Concrete two main choices exist. The International Commission on Large Dams (ICOLD) recommends within its Bulletin 51 [87] the use of blastfurnace (BLF) or pozzolan (POZ) cement since these types of cement have a stronger resistance against chemically aggressive water, as is also common knowledge within concrete technology [79, 105, 129]. In concrete construction it is also known, that through the use of BLF cement, concrete strength development at early age is much slower than with ordinary Portland cement [33, 79, 105, 114]. With this, when the secondary slurry-trench element is cut between two previously tremie-placed primary elements, the tremie concrete is still of low strength. This in turn makes it possible for trench cutters to cut the secondary elements precisely and with low wear for the cutter heads. However, the slow concrete strength development can also be counter productive if not controlled, since very low concrete strengths may cause construction operations to be halted before the secondary element can be cut. In this case, other blends or a low strength ordinary Portland cement can be used to achieve similar compressive strengths at 28 days but higher strength within the first days, irrespective of Plastic Concrete permeability. In addition, the regional availability of BLF or POZ cements may also be a limiting factor when choosing the cement type to be used.

Some studies on the structure of single-phase diaphragm wall materials (such as cement-bentonite mixtures) have however also shown that there is a difference within the material structure and mechanical parameters depending on the cement type used [55, p.22]. When using ordinary Portland cement the structure is more open than with blast-furnace cement, which is identifiable by SEM imagery shown in Figure 2.1 [55, p.24ff.].



(a) Ordinary Portland cement (b) Blast-furnace cement

Figure 2.1: SEM images of the micro-structure of hardened cement-bentonite mixtures depending on cement type used [55]

These images also show that, whilst the cement particle hydration occurs similarly in cement-bentonite mixtures and standard concrete, the mean particle distance increases from 2 μm for standard concrete to 15 μm for cement-bentonite mixtures [55, p.26]. The authors ascribe this to the increased w/c-ratio, but also to the presence of bentonite particles within the cement particle gap. They also hypothesise that formation of C-S-H occurs differently depending on the cement type used; with ordinary Portland cement, the high concentration of Ca^{2+}-ions at the cement particles causes C-S-H to precipitate at the clinker particles. On the the other hand, in blast-furnace cement the presence of slag particles further increases the cement particle distance, causing the Ca^{2+}-ions to be more evenly distributed. The authors therefore hypothesise that C-S-H may also form at the bentonite platelets, causing the platelets to be adhered together [55, p.27]. Test results showed hereby that the cement-bentonite strength increases and permeability decreases, with this effect being further reinforced the higher the slag content is [55, p.19ff.]. However, to date, this hypothesis remains unconfirmed. In addition, the interaction between cement and bentonite particles in Plastic Concrete, i.e. in the presence of aggregates and further admixtures, remain unexplored and should be subjected to further study.

2.1.2 Bentonite

Bentonite is a weathered rock composed of clay-like minerals which was first discovered in 1898 in Fort Benton, MT (U.S.A.) and is an alteration product of volcanic ash [78]. Although the bentonite discovered in Fort Benton is mainly composed of montmorillonite minerals ($\geq$ 80 wt-%), the term bentonite is however now well established and encompasses any clay-rock composed of smectite minerals, which in turn dominate the physical properties of the rock [78, p.1f]. The oven-dry density of bentonite generally ranges between $\rho \approx 2.65$ - 2.75 g/cm^3 [124, p.276].

Structure & Properties

Smectite minerals form platelets composed of three layers. The most common smectite mineral, montmorillonite, consists of two SiO_4-tetrahedron on opposite sides of a AlO_6-octahedron [112, 128]. Due to the partial, isomorphic substitution of some cations a layer charge is generated. This negative layer charge is in turn counter-balanced by other cations within the interlayer space between two adjacent platelets. Most commonly the interlayer cations are Ca^{2+}, Mg^{2+} or Na^{+} which neutralise the negative surface charge, and account for the two main bentonite groups Na-bentonite and Ca-bentonite (which commonly includes magnesium-bentonites) [90] [97, p.175f.] [112, p.10]. These interlayer cations do not however form ionic bonds but are instead bound through van-der-Waals interactions [106, p.1992]. In Figure 2.2 a schematic illustration of the montmorillonite structure is given, whereby the SiO_4-tetrahedrons and AlO_6-octahedron as well as interlayer cations (red) can be seen.

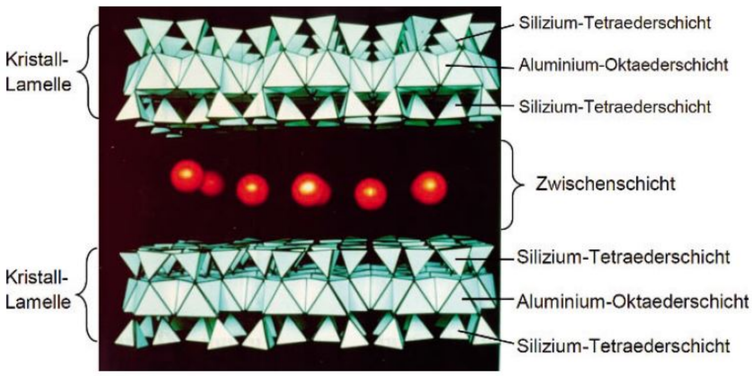


Figure 2.2: Schematic illustration of montmorillonite structure [112]

Furthermore, the weak layer charge permits the interlayer cations to adsorb and retain water molecules [85, 97]. The water adsorption capacity of sodium and calcium bentonite is however disparate, with Ca-bentonite adsorbing 200-300% water, while Na-bentonite can adsorb up to 600-700% of water [76, 112]. Moreover, water adsorption does not occur instantaneously. Ca-bentonite adsorbs most water within the first minutes [78, 85]. On the other hand, Na-bentonite adsorbs water more slowly, whereby water adsorption (also called hydration in the support fluid industry) is still not completed after 18 h [78, p.241]. Water is not however solely bound around interlayer cations, but water molecules also adsorb on to the negatively charged surface of the SiO_4-tetrahedrons [96]. The adsorbed water is hereby submitted to high surface tension ($\geq$ 2000 MN/m^2) and is strongly hindered from any movement [96, p.216]. The surface tension of adsorbed water, together with the low particle size and hydrated interlayer cations, causes bentonite and bentonite-composites to have a low permeability, since the water is hindered from transportation by the corresponding bonds [96, p.217]. The aforementioned water adsorption phenomena cause the clay minerals, especially montmorillonite to significantly increase in volume, multiplying its starting volume manifold. Due to the differing water adsorption capacity, Ca-bentonites exhibit a smaller swelling potential with the interlayer gap increasing up to a maximum value of 2 nm [85, p.I-6]. Na-bentonites on the contrary may swell far more strongly with the interlayer water even separating the individual platelets from one another [85, p.I-7].

Most naturally occurring bentonites predominantly have calcium ions in the interlayer space and are far more abundant than the more active Na-bentonites. However, Ca-bentonites may be "activated" to obtain Na-bentonites by exchanging Ca-cations with Na-cations within the interlayer space, enhancing material performance [97, p.176]. This most naturally occurs by mixing soda ash (Na_2CO_3) to Ca-bentonites with the partial precipitation of calcium carbonate ($CaCO_3$), although other technical procedures exist [90, 112]. This process is called alkaline activation, and is thoroughly described in Jasmund & Lagaly [90, p.363f.]. The activated Na-bentonites also have a more constant product quality than natural Na-bentonites due to this procedure [112, p.20] and may also be economically obtained with the appropriate process technology [106]. This cation exchange is however almost completely reversible and may occur when the bentonite is put in contact with the released Ca^{2+}-ions from cement [97, p.181], causing cement-bentonite mixtures to flocculate and become unstable [85]. Therefore for cement-bentonite applications a cement-stable bentonite is required, which maintains its rheological, swelling and mechanical properties in the presence of cement particles, allowing for the production of stable cement-bentonite mixtures [97, 107]. Although some contemporary, commercially available bentonites have proven compatible with cement in some applications, the reasons for bentonite cement-stability are not yet fully understood. Some authors suggest the content of free soda-soluble silica or the presence of accessory minerals in bentonite may be a determining parameter for cement-stability [107]. It is therefore imperative to further study the cement-bentonite interaction to establish the reigning interaction mechanisms [84].

Characterisation

At present, no satisfactory specifications for bentonite for slurry trench works exist [86, 93]. Solely the OCMA specification DFCP-4 for drilling fluid material bentonites is sometimes cited, does however not fulfill slurry trench requirements [86]. For this reason, standard geotechnical testing procedures are commonly used to characterise commercially available bentonites [92, 93]. Bentonite as a mineral or soil, can therefore be characterised using the Atterberg limits, whith testing following DIN 18122-1 [57] or ASTM D4318 [17]. The Atterberg limits are a basic measure of the water content of a fine-grained soil. The limits most commonly measured are the plastic limit (w_p) and liquid limit (w_l). The difference of these two limits, is the so called plasticity index (I_p) and defines the range size of water contents where the soil exhibits plastic properties [32, p.107f.]. In Figure 2.3 an overview of the soil states and corresponding water contents is given. For example, regular silt has a plasticity index I_p in the range of 10, whilst clay usually lies at around 30 [117, p.54]. On the contrary bentonite commonly has a plasticity index of 100 with a liquid limit close to 150, far exceeding regular soils [78, p.239].

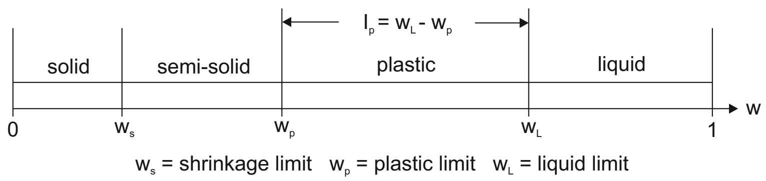


Figure 2.3: Overview of soil states and corresponding water content

Another parameter is the so called activity index (I_A), which is defined as the ratio of plasticity index (I_P) to the percentage of clay-size particles (i.e. particles with d < 0.002 mm). In Equation 2.1 the activity is given. The activity of a given soil may shed light on the minerals contained within the soil [74, p.357f.] [117]. Regular kaolinite has an activity index of approximately 0.4, Ca-bentonite of approximately 1.5 and Na-bentonite often reaches values greater than 7 [78, 117].

The presence of clay minerals can be confirmed with X-ray diffractometry. In addition, the amount of clay present in aggregate can be determined using the methylene-blue test, which also measures the cation-exchange capacity of soils [111, p.290]. Some authors also recommend using the methylene-blue test to determine the montmorillonite content within a bentonite sample, since this is also decisive for the bentonite properties [55, p.11].

$$I_A = \frac{I_P}{m_T/m_d} \tag{2.1}$$

where			
	I_A :	activity index	(-)
	I_P :	plasticity index	(-)
	m_T :	dry mass with d < 0.002 mm	(g)
	m_d :	dry mass with d < 0.4 mm	(g)

Furthermore within the context of specialist foundation construction it may be advisable to test the bentonite swell index as well as the corresponding betonitic slurry free-fluid fraction. The former is tested following ASTM D5890 [19] within laboratory conditions and enables the evaluation of swelling properties of any clay mineral which may be used for hydraulic conductivity reduction. The free-fluid volume fraction is determined following DIN EN ISO 10426-2 [51] and is useful to understand the static stability of the bentonite or cement-bentonite slurries.

Last but not least the water adsorption capacity of bentonite is most commonly measured following DIN 18132 [60] for soils and DIN EN ISO 10769 [52] specifically for bentonite. In the latter, the previously dried bentonite specimen is placed on a filter and the adsorbed water is measured over a defined period of time.

Application

Due to the previously mentioned structure, bentonite is highly swellable and thixotropic and has therefore manifold industrial applications especially in the construction industry. Most commonly bentonite has been used in the specialist foundation construction when placing slurry trench walls, where a bentonite-suspension is used to hydrostatically stabilise the adjacent ground [124]. Furthermore, bentonite is used as a stabilising agent in drilling fluids and cement suspensions improving workability and reducing the risk of segregation [78, 103, 113]. Some authors also use bentonite to waterproof soil and structures, reducing the overall permeability [74, 78, 113]. Finally, bentonite is also used in regular and nuclear waste disposal, since the high cation exchange capacity of montmorillonite enables bentonite to adsorb chemical pollutants and heavy metals, hindering their passage through a seepage barrier [85, 96].

Bentonite has historically been used in Plastic Concrete as it was a commonly available, cheap stabilising agent for Plastic Concrete mixtures [73, 76]. In addition bentonite allows for a more ductile behaviour of a Plastic Concrete diaphragm wall [74, 87, 89, 125]. Some authors suggest that through bentonite swelling the air voids within the cement paste structure are filled partially and the permeability is reduced [74, 76, 118]. This however only likely happens if the bentonite has not previously been completely hydrated, allowing for a further bentonite hydration in the hardening Plastic Concrete. However, the mixing sequences in Plastic Concrete production are manifold and heavily depend on construction site specifications, hence a full or partial hydration can not be safely assumed (see section 2.3).

Scholz et al. studied the mechanical properties of various diaphragm wall compounds and stated that the average pore size within Na-bentonite specimens is notably smaller than those within Ca-bentonite specimens [118]. In addition, the bentonite structure varies, with sodium-bentonite presenting a face-to-face structure, whilst calcium-bentonite shows a combined face-to-face, edge-to-edge structure. Calcium-betonite has gel pores in addition to capillary pores [118]. This in turn affects the permeability of bentonite-mixed materials, since for example small amounts of bentonite significantly reduce the permeability of sands. Na-bentonite mixed sands have a far lower permeability than Ca-bentonite mixed sands with difference being two orders of magnitude [78, p.243].

2.1.3 Aggregates and Admixtures

The most important criteria for the choice of aggregates in Plastic Concrete is the maximum grain size, due to the high segregation risk of fresh Plastic Concrete. This is caused by the relatively high w/c-ratio and the need to use bentonite as a stabilising agent (see section 2.2). Therefore, aggregates are generally limited to sands and fine to medium gravels. According to the DWA guideline M512-1 a maximum grain size of $d_{max} = 63\,\mathrm{mm}$ should be adopted [56, p.60]. Similarly ICOLD Bulletin 51 recommends a maximum grain size of $d_{max} = 30\,\mathrm{mm}$ and does not contain too large a fraction of fines [87, p.29]. Practical examples and experience have however shown that such mixtures show very strong segregation effects, and therefore smaller grain sizes should be used. Most commonly a maximum grain size of $d_{max} \leq 12\,\mathrm{mm}$ is used for Plastic Concrete mixtures. The USBR Design Standard 13(16), recommends maximum grain size to be limited to $d_{max} \leq 25\,\mathrm{mm}$ [125, p.59]. The Austrian standard ÖNORM B4452 [108] limits the maximum grain size to $d_{max} \leq 22\,\mathrm{mm}$, however notes that maximum grain size above 16 mm is rare. The standard also states that special consideration should be given to the segregation risk and deformability of Plastic Concrete mixtures when using a maximum grain size $d_{max} \geq 8\,\mathrm{mm}$. Furthermore, the fine particle content is also partially regulated to guarantee the necessary flowability [108, p.4]. It should however be noted that it is often difficult to meet specific grading demands at building sites in some countries. Furthermore, rounded aggregate is preferred as this type of aggregate further enhances the flowability of tremie concrete [130]. Moreover, the type of aggregate used is regulated by the exposure to any aggressive contaminant, with quartz based aggregate being the preferred aggregate type [130]. Occasionally, additional mineral fillers (i.e. stone dust, clay dust and fly ash) are added to Plastic Concrete mixtures to further increase the solids content, although this is not a commonly used procedure [85, p.VIII-9].

Various types of admixtures are also used in Plastic Concrete mix designs. Most often, retarding admixtures are used to slow down concrete setting and prevent premature concrete stiffening [125, p.57]. With this a longer workability window is achieved and longer slurry trench elements can be produced, for which concrete placement with the tremie method can be safely finalised. Depending on Plastic Concrete mixture composition varying amounts of retarding admixtures may be added normally ranging from 1 wt-% to 2.5 wt-% of cement content [24, p.47]. Especially with long slurry-trench elements, the retarding agent dosage has to be carefully measured, since a shortfall of retarding agent can cause the first concrete batches to stiffen and then in turn be displaced like a plug above the fresh tremie placed concrete [24, p.47].

In some cases superplasticizing admixtures are also used to ensure better and more controlled workability of the Plastic Concrete mixture. It should however be noted that the effectiveness of modern polycarboxylate ether-based superplasticizers (PCEs) is negatively affected by the presence of clay minerals, especially montmorillonite [111, p.289]. Therefore, corrective actions should be taken when using PCEs (e.g. the use of PCEs with hydroxyalkyl side chains) [111, p.290]. Exceptionally further stabilising admixtures are used to stabilise the fresh concrete if the bentonite proves to be insufficient.

In most instances tap water is generally suitable for Plastic Concrete production. However, untreated water or water with high ion concentrations may affect bentonite dispersion or hydration process and should therefore be tested in trial mixtures if required [86, p.N19].

2.2 Mixture Composition

As mentioned previously, Plastic Concrete is also a five-phase material. In contrast to standard concrete, the w/c-ratio of Plastic Concrete is much higher, with values ranging from 3.3 to 10 [87]. The cement content is also significantly lower than that of standard concrete, rarely surpassing the 200 kg/m^3 mark and even being as low as 80 kg/m^3. In addition, Plastic Concrete contains bentonite with the bentonite content laying anywhere between 0.5% and 12% by weight of dry mass of constituents depending on the mixture composition and bentonite type. Generally speaking, the lower bentonite contents correspond to Na-bentonite, whilst the higher contents correspond to Ca-bentonite, which is caused by the differing swelling behaviour. Some authors suggest a Na-bentonite to Ca-bentonite equivalence of 1 : 5, suggesting a more economic approach through the use of Na-bentonite [55, p.15]. Furthermore, Plastic Concrete contains somewhat smaller similar quantities of aggregate to standard concrete, hereby ranging from 1100 kg/m^3 to 1500 kg/m^3.

In Figure 2.4 five different concrete mixtures are shown, of which three correspond to Plastic Concrete mixtures. The standard concrete example from Grübl et al. [79] represents a standard concrete with 20 MPa strength at 28 days. The middle three mixtures are examples for Plastic Concrete with an approximate compressive strength of 1.30 MPa at 28 days [22, 116]. Finally, a mixture composition by Triantafyllidis [124] of a single-phase diaphragm wall material with 1 MPa compressive strength at 28 days is given for comparison.

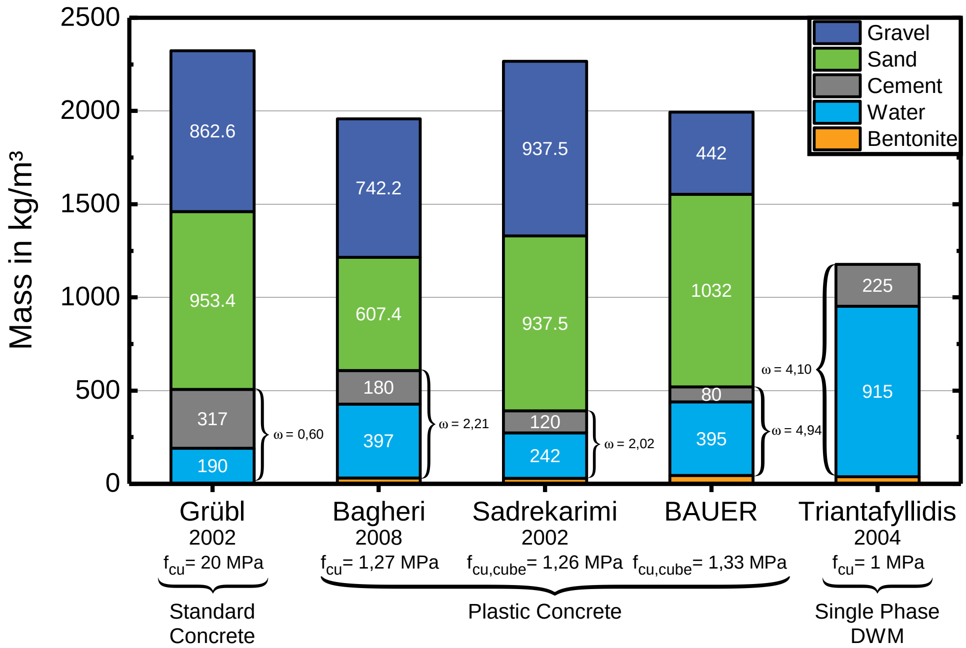


Figure 2.4: Representative examples of Plastic Concrete mix designs

As can be seen, the Plastic Concrete mix design is a combination of standard concrete and single-phase diaphragm wall material. The use of aggregates (most notably sand and fine gravel) in somewhat reduced quantities compares to the composition of standard concrete. The density of Plastic Concrete is also similar to that of concrete ranging from 1.9 g/cm^3 to 2.3 g/cm^3. The Austrian standard ÖNORM B4452 requires a minimum density of 1.80 g/cm^3 [108, p.9], since a density difference of 0.5 g/cm^3 is necessary to effectively displace the bentonite slurry within the slurry trench element without mixing, when placing concrete with the tremie method [108, p.14] [124]. Other authors recommend a density difference of 0.75 g/cm^3 [124, p.270]. The w/c-ratio on the other hand compares to that of single-phase diaphragm wall materials, exceeding by far 1.0 implying the existence of a far coarser micro-structure. Also, the use of bentonite as a stabilising admixture is comparable to that of single-phase diaphragm wall materials. It should be emphasised that single-phase diaphragm wall materials are not placed with the tremie method and therefore is also subjected to other construction uncertainties and limitations [74, 124].

2.3 Mixing Sequence

Across the literature the Plastic Concrete mixing process is not consistent. Various options are presented, which are schematically shown in Figure 2.5.

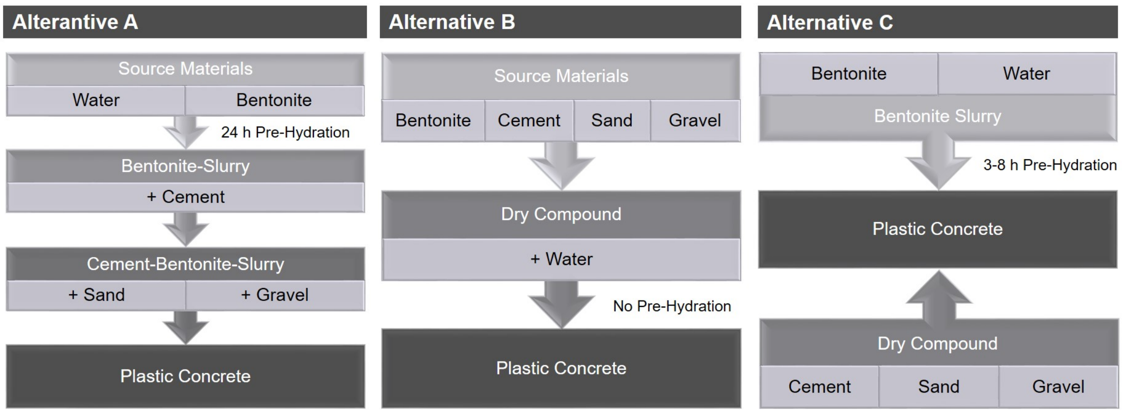


Figure 2.5: Representative examples of Plastic Concrete mixing sequences

Alternative A is the most commonly described variant in literature [22, 28, 72, 80, 89, 101, 110, 116]. In this bentonite and water are gradually mixed together and the let to hydrate for up to 24 h. After this, cement is added to the bentonite-suspension and thereafter the aggregates are added. Alternative B [135] pre-mixes the components bentonite, cement and aggregates to a dry compound. This compound is then mixed with water and placed within the slurry-trench element without allowing for any hydration time. This method is however not commonly used in practice, since the dry mixing of components and subsequent water addition does not achieve a sufficiently homogeneous Plastic Concrete mix [95, p.240]. Finally, alternative C is an often used mixing sequence by some construction companies, whereby cement and aggregate are mixed to a dry compound, whilst bentonite and water are mixed into a slurry [95]. The bentonite slurry is then mixed with the dry compound to obtain the Plastic Concrete mixture, whereby the bentonite slurry is not allowed to hydrate before use. Alternative C can therefore be considered a combination of alternative A and B. It should be noted however that due to the differing mixing sequence and hydration time, varying results can be expected in terms of mechanical properties and permeability values. This is most likely the fact, as the bentonite hydration phase is different for the three aforementioned alternatives, which in turn affects the void filling in the hardened cement paste.

The hydration of bentonite is however not only dependant on the aforementioned differences between bentonite types (see subsection 2.1.2) but also on the type of mixer and thus the induced shear rate $\dot{\gamma}$. For any given mixer it can be seen that the higher the maximum achievable shear rate $\dot{\gamma}$ is, the shorter the hydration time required for bentonite samples will be [55].

2.4 Fresh Properties

To ensure the correct placement of concrete, which in turn enhances hardened concrete quality, the fresh properties of Plastic Concrete mixtures have to be controlled, especially concrete flowability during the whole casting process. Therefore, the fresh properties must be controlled not only during inital placement, but also measure the thixotropic and flow retention characterestics of the concrete [98, p.215f.]. The flow properties are mainly affected by concrete rheology which in turn result from the concrete mix design [98]. Despite the complexity and relevance of concrete rheology it is still not uncommon for simple concrete testing procedures to be used to determine the fresh properties of concrete [98, p.211]. It should be noted that many problems in diaphragm walls may be attributed to the use of ianadequate concrete mixes resulting from poor concrete specifications due to deficient or simplistic testing procedures [70, 98]. Most commonly the so called slump test [38] and flow table test [39] are used, although other tests do exist. In Table 2.1 an overview of the most common testing procedures for fresh concrete flowability and their corresponding testing standard is given.

Table 2.1: Overview of common testing procedures for fresh concrete workability

German	DIN-Standard	English	ASTM-Standard
Setzmaß	EN 12350-2	slump	C143/C143M
Setzfließmaß	EN 12350-8	slump-flow	C1611/C1611M
Ausbreitmaß	EN 12350-5	flow table	-
L-Kasten	EN 12350-10	L-Box	-

For Plastic Concrete placed with the tremie method various guidelines and standards exist, which require specific values of concrete fresh properties. According to DIN EN 1538 [47] which in turn refers to Appendix D of DIN EN 206 [50] the flow table test values for concrete placed with the tremie methods should be 600 mm. Alternatively, a slump test can be performed with a target value of 200 mm. Similarly, the Austrian standard ÖNORM B4452 [108] requires the concrete to obtain flow table test values in the range of 55 cm to 65 cm. This standard also limits the free-fluid test value following DIN EN ISO 10426-2 [51], commonly used to test sedimentation stability, to 2% after 2 hours [108, p.9]. The DWA guideline M512-1 [56] recommends a flow table test value greater than 530 mm for Plastic Concrete. Similarly, in USBR Design Standard 13(16) a 15 cm to 23 cm slump is desired to ensure a high degree of fluidity and workability [125, p.56].

Furthermore, the density should be measured following EN 12350-6 [40] when the Plastic Concrete dosing is volumetric.

Other tests such as for example the L-Box test following DIN EN 12350-10 [37], although developed for super-workable concrete, is not entirely adequate for Plastic Concrete since the high flowability and low maximum aggregate size makes the containment of the concrete within the L-Box difficult. This in turn does not allow for the calculation of the passing ability ratio (PL). For a more detailed investigation into the application of the L-Box test for tremie pipe concrete reference is made to [1]. Some guidelines also refer to the Marsh funnel viscosity following DIN 4127 [66] or ASTM D6910 [20], when determining the fresh concrete flowability. It should be noted however that a Marsh funnel has a maximum opening at the bottom of 4.75 mm and a entry screen of 3.2 mm, which in turn only really allows for the measurement of bentonite-slurries without aggregate. Nonetheless, ICOLD Bulletin 51 requires a Marsh funnel viscosity of 50 s for the bentonitic slurry [87, p.27]. Other testing methods for Concrete workability for Deep Foundations exist (e.g. Modified Cone Outflow test) and may be reviewed in [71, Apx.A].

Concrete flowability is generally controlled through the water content and superplasticizing agent content, however the stability of the Plastic Concrete have to be closely monitored. Evans et al. [72] also suggest, that the workability of of Plastic Concrete is enhanced through the addition of fly ash, due to its ball bearing type action.

For more detailed information regarding the various fresh concrete testing methods applicable for tremie method refer to the *EFFC/DFI Guide to Tremie Concrete for Deep Foundations* [71]. Scientific fundamentals on concrete rheology can be found in [115].

3. Mechanical Behaviour

Especially important is the precise description of the mechanical behaviour of Plastic Concrete. For this various testing methods and standards exist to study the mechanical properties, which are described in section 3.1. In section 3.2 the mechanical strength of Plastic Concrete using the aforementioned testing methods is described. The deformation properties of Plastic Concrete (e.g. Young's modulus) are set out in section 3.3. Finally, the relaxation potential of Plastic Concrete is reported in section 3.4.

3.1 Testing Standards

Plastic Concrete has a relatively low compressive strength at 28 days in the range of 1 MPa to 3 MPa. This in turn causes Plastic Concrete to have a very low compressive strength for concrete testing methods, but a rather high compressive strength for soil testing methods. Plastic Concrete can therefore be considered to be in the transition zone of these testing methods.

3.1.1 Compressive Strength

The most common testing methods for concrete strength is the unconfined compressive stress (UCS) test. In this concrete samples are placed within as testing machine and axially loaded. The standard European test method is EN 12390-3 [44], the corresponding ASTM Standard is ASTM C39/39M [9].

For the testing of soil strength various testing methods are used. Firstly, the UCS is measured in a similar manner to that of concrete, following DIN 18136 [62] or ASTM D2166/2166M [14]. However, for soil various other shear strength and triaxial compressive strength test methods exist. Triaxial testing is herein divided into three types of triaxial tests; Consolidated Drained (CD), Consolidated Undrained (CU), Unconsolidated Undrained (UU). The main diferences between the three types, lie within the consolidation state of specimens and the build up of pore water pressure. Consolidated specimens are previously loaded until the measured total stress σ equals the measured effective stress σ', i.e. that the pore water pressure $u = 0$, as shown in Equation 3.1. In drained experiments water can gradually exit the specimen and therefore no pore pressure build up exists ($\Delta u = 0$). In undrained experiments the system is "closed" with which the pore pressure increases ($\Delta u \neq 0$) and is measured during CU tests [117]. It should be however noted that for the practical measurement of Plastic Concrete the pore water pressure is of negligible influence.

$$\sigma' = \sigma - u \tag{3.1}$$

where	σ' :	effective stress	(MPa)
	σ :	total stress	(MPa)
	u :	pore water pressure	(MPa)

In Germany all triaxial tests summarised in one standard and performed following the DIN 18137 testing standard [63, 64]. The ASTM standards divide the three testing methods CD, CU and UU into the three standards ASTM D7181-11 [21], ASTM D4767-11 [18] and ASTM D2850-15 [16], respectively.

It is also interesting to note that geotechnical standards solely test cylindrical samples with varying height-to-diameter (h/d) ratios. Concrete standards allow for compressive strength testing on cylindrical samples with h/d = 2, but also on cubic samples which is of common practice. This should be considered, since both the h/d-ratio as well as the specimen shape affect the test results obtained, but no direct conversion formulae exist [105, p.596]. In concrete technology it is of common knowledge that cubic specimens have a higher strength than cylindrical samples, and an increasing h/d-ratio further decreases the tested strength [79, 109, 114]. Cylinders are believed to give a greater uniformity of results as their strength is less affected by coarse aggregate properties, lesser end restraint influence and more uniform stress distribution in horizontal planes [105, p.596].

Some authors also use other, non-destructive testing methods to estimate the compressive strength. For cement treated soils, various authors have related the compressive strength of cement-treated soils to the small strain shear modulus measured with bender elements [119, 127]. However, these testing methods are not standardised for concrete testing and can therefore only be considered for compressive strength estimation.

3.1.2 Elastic Modulus

On the contrary, the Elastic Modulus E is defined and tested very differently depending on the field of study. For concrete, the elastic modulus is defined as the secant modulus, i.e. the slope between two points where Hooke's law is applicable. This modulus is of utmost importance, since it is typically used for analysis purposes [125, p.58]. Testing is performed following DIN EN 12390-13 [41] or ASTM C469 [10], whereby the elastic modulus is determined between two pre-set stress levels ($\sigma_a = f_c/3$ and $0.10 \cdot f_c \leq \sigma_b \leq 0.15 \cdot f_c$). The necessary stress is applied through the pressure plates of the testing machine while strain is most commonly measured using strain gauges or linear variable differential transformers (LVDT). For more information regarding strain measurement possibilities refer to [81]. In Equation 3.2 the elastic modulus definition following DIN EN 12390-13 [41] is given.

$$E_{C,0} = \frac{\Delta\sigma}{\Delta\varepsilon_0} = \frac{\sigma_a^m - \sigma_b^m}{\varepsilon_{a,1} - \varepsilon_{b,0}} \tag{3.2}$$

where	$E_{C,0}$	:	initial elastic modulus	(MPa)
	σ_a^m	:	measured upper testing stress	(MPa)
	σ_b^m	:	measured lower testing stress	(MPa)
	$\varepsilon_{a,1}$	:	measured strain at the upper testing stress	(-)
	$\varepsilon_{b,0}$	:	measured strain at the lower testing stress	(-)

On the other hand, the testing of soil deformability has manifold possibilities. The most common testing methods are the unconfined compression test and Oedometer consolidation test. Within the unconfined compression test, the standard defines a deformation modulus as the "modulus of the uniaxial compression test (E_u)" which is determined from the maximum tangential slope of the stress-strain line following DIN 18136 [62]. DWA guideline M512-1 [56] for example reycommends the use of E_u as the defining parameter. ASTM D2166/2166M [14] on the other hand uses the initial tangent modulus for this measurement. It should be noted however that the strain measurement is not performed directly on the specimen, but instead determined from the measurement of the piston movement. This most likely also incurs in the measurement of machine displacement which in-turn provides less accurate strain measurements of the Plastic Concrete specimens [82]. Alternatively, soil deformability can also be measured with the Oedometer consolidation test following DIN 18135 [61] or ASTM D2435/2435M [15]. In this, a specimen is compressed within a confining ring by imposing a load over a frame in drained conditions ($\sigma = \sigma'$). The sample compression is then subsequently measured over time by a dial indicator. With the resulting stress-settlement curve the Oedometer modulus E_s (German: Steifemodul) can be calculated with Equation 3.3 [117, p.127ff.].

$$E_s = \frac{\Delta\sigma'}{\Delta\varepsilon} \tag{3.3}$$

where E_s : Oedometer modulus (MPa)
$\Delta\sigma'$: effective stress increase (MPa)
$\Delta\varepsilon$: strain increase (MPa)

It should be noted however, that the Oedometer modulus is dependant on the stress range used and is measured in confined conditions, unlike the aforementioned moduli E and E_u.

A tabular listing with DIN standards and corresponding ASTM standards for the aforementioned testing methods is given in Table 3.1.

Table 3.1: DIN and ASTM Test Method Standards Comparison

Field	Test	DIN-Standard	ASTM-Standard
Concrete	Compressive strength (UCS)	EN 12390-3 [44]	C39/C39M [9]
	Young's modulus	EN 12390-13 [41]	C469/C469M [10]
	Triaxial Tests	-	C801* [13]
Soil	Compressive strength (UCS)	DIN 18136 [62]	D2166/D2166M [14]
	Oedometer consolidation	DIN 18135 [61]	D2435/D2435M [15]
	Triaxial Tests - CD	DIN 18137 [63, 64]	D7181 [21]
	Triaxial Tests - CU	DIN 18137 [63, 64]	D4767 [18]
	Triaxial Tests - UU	DIN 18137 [63, 64]	D2850 [16]

* = standard withdrawn

All in all it should be recommended to use concrete testing standards, since Plastic Concrete composition (see section 2.2) and properties correspond to those of concrete materials. In addition, the direct strain measurement of samples (e.g. with strain gauges, LVDTs, etc.) is much more precise than machine displacement measurements. However, none of the aforementioned standards is ideal since Plastic Concrete's strength is very low compared to standard concrete and rather high for soil, which in turn requires an adjustment testing parameters (e.g. loading speed, specimen preparation, strain measurement, etc.). Furthermore, the linear-elastic stress-strain relationship implied by Hooke's law is also not necessarily applicable for Plastic Concrete samples [92]. DWA M 512-1 [56] guideline recommends for example that the UCS should be tested according to DIN EN 12390-3 [44] for an expected UCS > 2 MPa and tested according to DIN 18136 [62] for an expected UCS < 2 MPa. It should be however noted that the real sample strength at a tested UCS of 2 MPa is not identical for both testing methods. It is therefore recommended that the exact testing conditions should therefore be specified during planning and tendering of projects to avoid testing induced differences of acceptance criteria [25, p.41].

3.1.3 Testing Influences

It is of common knowledge in concrete testing, that various influencing factors exist which affect testing results. Most notably the loading speed, but also sample preparation and storage conditions can affect the obtained test results. An overview regarding the various influencing factors on concrete strength testing can be found in [109]. In the following the effect of some of these influencing factors are described in more detail against the background of Plastic Concrete specimens.

Specimen end conditions

The ASTM standard practice for bonded capping of cylindrical concrete specimens is ASTM C617 [12]. In Germany the capping process of concrete specimens is embedded within the appendix A of DIN EN 12390-3 [44]. Herein four different methods are anchored namely sanding, calcium aluminate cement mortar capping, sulphur mortar capping and the sandbox method. This standard states that the application of calcium aluminate cement mortar and sulphur mortar is limited to specimens with an expected strength of 50 MPa. This is due to the fact that the mortar pastes have a relatively low elastic modulus and therefore deform significantly when high loads are applied to test high strength concrete [53, 54]. Generally, capping materials should be at least as strong as the concrete they are bonded to [109]. In addition the capping layer thickness should be kept to a minimum, as Dahm et al. [53, 54] showed that a greater layer thickness changes the state stress within the specimen. The authors also note that cylindrical specimens are affected less by specimen capping variations than cubic specimens, exhibiting less scatter in the test results. For Plastic Concrete specimens it should be however noted that these specimens have a very low strength. For this reason specimen capping with mortar application should be unrestrictedly possible. In addition the literature review shows that various capping methods are applied, without any single specimen standing out [80, 93]. It should also be mentioned that due to the high w/c-ratio there is a relatively high sedimentation potential. In the event of sedimentation Plastic Concrete specimens exhibit a water-rich layer in the upper specimen section, which in turn incurs in low concrete strength. It is therefore necessary to account for possible sedimentation when reviewing Plastic Concrete test data.

It is also questionable however, whether specimen capping is actually necessary for Plastic Concrete samples. During the loading of concrete specimens with uneven ends, critical stress peaks can occur which reduce the measured concrete strength. The high relaxation behaviour of Plastic Concrete however likely incurs in the reduction of critical stress peaks. It remains to be shown, whether specimen sanding or cut-off may affect Plastic Concrete structure by inducing strength-reducing cracks and stresses.

Loading speed

It is common knowledge that concrete is a crack afflicted material [114]. Therefore with increasing loading speed, the measured concrete strength increases as the possibility of crack propagation around aggregate particles is reduced favouring particle rupture [109, 114]. At very high rates of loading additional inertial effects may occur [114]. At very low loading speeds, creep deformation may also occur in addition to elastic deformation, causing concrete testing to determine lower compressive strength [79, 109, 114]. Concrete has a permanent load resistance (the so called creep strength) of 70% of the short-term resistance at a loading speed of 0.2 MPa/s [105].

In DIN EN 12390-3 [44] the loading speed for concrete specimens is set to 0.6 ± 0.2 MPa/s. It should be noted that at this loading speed standard concrete mixtures take 60 to 90 seconds to reach there peak load and rupture. The National Annex of DIN EN 12390-3 also notes that at compressive strengths above 80 MPa or below 20 MPa the loading speed may be adjusted.

ASTM C39/C39M [9] establishes that a rate of movement corresponding to a specimen stress rate of 0.25 $\pm$ 0.05 MPa/s should be applied. During the first half of the anticipated loading phase, a higher rate of loading is permitted, the concrete specimen should not however be subjected to shock loading.

On the other hand, geotechnical testing standards for soil DIN 18136 [62] and ASTM D2166/D2166M [14] use strain rate as the defining loading speed. DIN 18136 [62] establishes a strain rate of 1% of the sample height per minute while ASTM D2166/D2166M [14] requires a strain rate between 0.5% and 2% of the sample height per minute.

Neville however states that within the practical range of loading rates (0.07 to 0.7 MPa/s), the measured strength of standard concrete varies only between 97% and 103% of the strength obtained at 0.2 MPa/s [105]. Neville also notes that stronger concrete exhibits lower sensitivity to the strain rate [105], which may suggest higher loading speed sensitivity for Plastic Concrete specimens. It should also be noted that standard concrete testing machines may be limited in their measuring precision, as the application of axial loads lower 40 kN may be subjected to high scatter or even impossible. Measurement accuracy should therefore always be ensured before testing [45].

According to Kazemian et al. [93] the stress-strain behaviour of Plastic Concrete differs from that of ordinary concrete (not linear between 0% to 40%) and, as expected, the standard loading speed is generally too high to measure stress-strain. Hinchberger et al.'s study [80] with strain controlled experiments also showed that Plastic Concrete is sensitive to compression rate, whereby higher compression rates (0.01 mm/min $>$ 0.001 mm/min) result in higher compressive stress values, as would be expected [80, fig.7].

It is therefore necessary to adjust the standard loading speed in concrete standard for Plastic Concrete specimens to achieve measurable and precise data, which is also in line with DIN EN 12390-3 [44] for specimens with compressive strength below 20 MPa. Normally, and in accordance with normal strength concrete testing procedures, specimen failure should be achieved within 60 s to 90 s, which means that a Plastic Concrete specimen with an expected 2 MPa strength, should be tested with a loading speed between 0.02 MPa/s and 0.03 MPa/s. For example, in DIN 4093 [65], which regulates the design of strengthened soil using jet grouting, deep mixing or grouting techniques, the loading speed is reduced to 0.05 MPa/s for samples with an expected compressive strength $f_{cyl,m} \leq$ 4 MPa. This loading speed would also be in line with Plastic Concrete requirements and achieve failure after approximately 20 s.

In Table 3.2 an overview of various testing methods for compressive strength determination and their corresponding loading speed is given.

Table 3.2: Compressive strength testing methods and corresponding loading speed

Standard	Type	loading speed
DIN 18136 [62]	strain rate	$(0.01 \cdot h_0)$/min
ASTM D2166/D2166M [14]	strain rate	$(0.005 \cdot h_0)$/min to $(0.02 \cdot h_0)$/min
DIN EN 12390-3 [44]	stress rate	0.6 ± 0.2 MPa/s
ASTM C39/39M [9]	stress rate	0.25 ± 0.05 MPa/s
DIN 4093 [65]	stress rate	0.05 MPa/s

h_0 = initial sample height

3.2 Plastic Concrete Strength

As mentioned in section 3.1 the mechanical behaviour of concrete samples is most commonly related to the samples' compressive strength. However for cut-off wall design the knowledge of Plastic Concrete's tensile strength as well as multi-axial strength is also of utmost importance. Therefore, in the following subsections Plastic Concrete's compressive, tensile and multi-axial strength will be discussed and placed within the concrete technology context.

3.2.1 Unconfined Compressive Strength (UCS)

General

The strength of Plastic Concrete can be characterised with various parameters, most commonly however the unconfined compressive strength (UCS) is therefore used. In concrete technology the w/c-ratio is the most common parameter affecting concrete strength, whereby a lower w/c-ratio incurs in higher concrete strength [79, 105]. Various studies have tested the UCS of Plastic Concrete with varying mix designs [2,22,28,72,80,93,101,110,116]. In Figure 3.1 an overview of the determined UCS from the aforementioned references is given. The experimental data plotted in Figure 3.1 corresponds to cylindrical Plastic Concrete specimens with a height-to-diameter ratio of 2.0 (with varying size) produced with common mixture compositions as described in section 2.2. The data shape indicates which testing standard was used as can be seen in the graph legend.

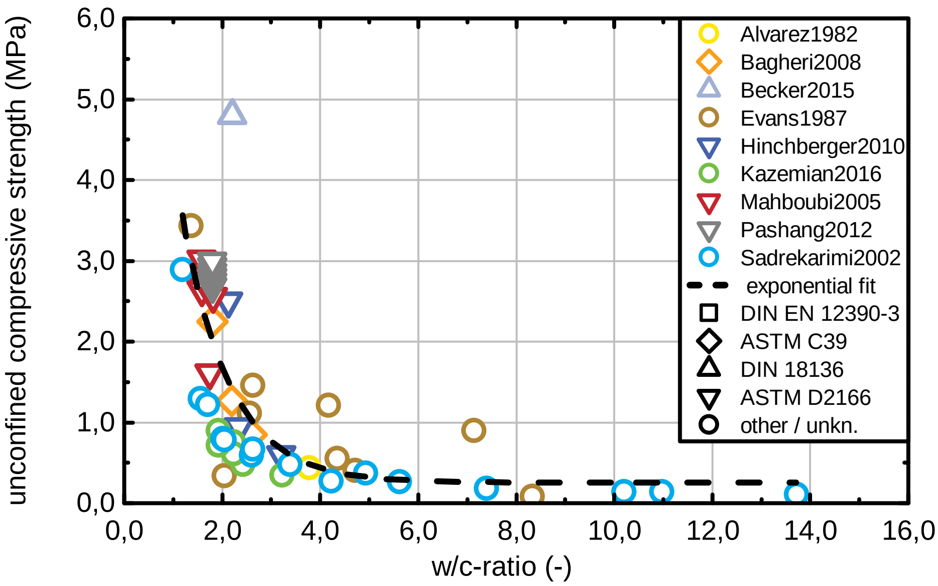


Figure 3.1: Overview of the UCS of Plastic Concrete as a function of w/c-ratio at 28 days

The graph shows that, as would be expected, there is a gradual decline in Plastic Concrete strength with increasing w/c-ratio, closely describing an exponential trend. In addition some authors [72, 116] use a very high w/c-ratio far exceeding commonly used w/c-ratios. However, due to the presence of bentonite the effective w/c-ratio is smaller, since the bentonite absorbs water into its structure reducing the readily available water for cement hydration. For this reason Geil defines a reduced water cement ratio w_{red}/c to account for the water binding capacity of bentonite [76, p.45]. The author does not herefore provide a mathematical formula, but instead provides a nomograph with which w/c-ratio can be reduced as a function of cement content, bentonite content and bentonite type [76, p.46]. It should here by again noted, that the water binding capacity of bentonite is different for Na-based and Ca-based bentonite, as described in section 2.1. The author however fails to analyse the contending behaviour of cement and bentonite for the available water and the likely interaction between these.

If water adsorption capacity of bentonite is (simplistically) assumed to be 2.0 (in-line with section 2.1) the reduced water content w_{red} can be estimated to $w_{red} = w - 2 \cdot m_{bentonite}$. With this, the overview in Figure 3.2 can be obtained.

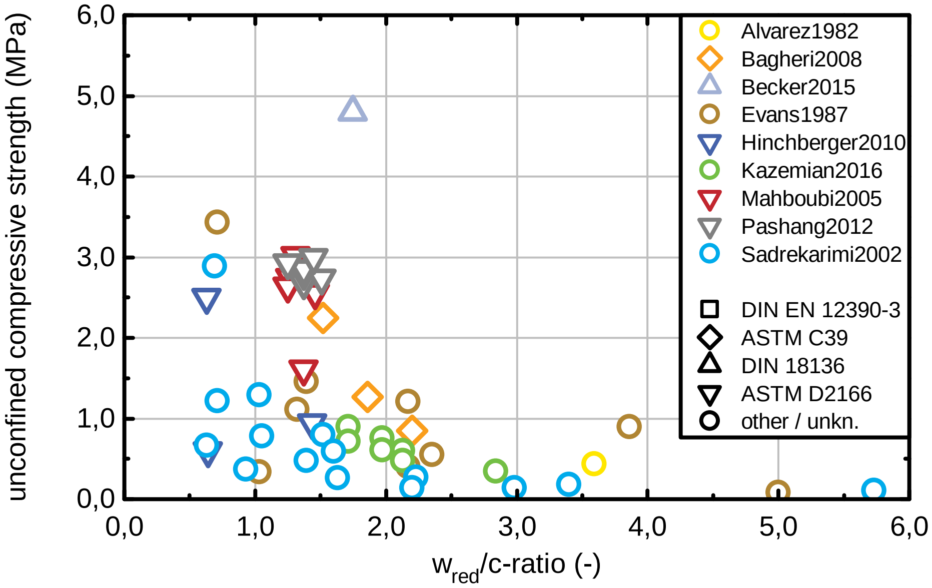


Figure 3.2: Overview of the UCS of Plastic Concrete as a function of w_{red}/c at 28 days

It can be seen, that the w_{red}/c relates to more realistic values ranging from 0.5 to 6.0. The expected exponential trend is no longer so clear for the data as a whole, however a slight decrease in unconfined compressive strength with decreasing w_{red}/c-ratio can be recognised when looking into single data sets. It does however stand out that, with some exceptions, the results obtained from geotechnical testing standards ASTM D2166/D2166M [14] and DIN 18136 [62] tend to be higher than those obtained from other testing standards, which implies a testing induced difference. It furthermore becomes clear that the aforementioned simplistic approach to w_{red} does not account for all influencing parameters (e.g. bentonite type, bentonite characteristics, cement type, etc.). Moreover the simplistic approach does not account for the effect of bentonite water adsorption and the contending behaviour of cement and bentonite, which should therefore be further analysed in more detail.

In concrete technology and design, standard concrete normally achieves a fracture strain of approximately 0.2% to 0.3% when tested under standardised unconfined compression conditions [79, p.378]. This guide value is however also dependant on loading speed, whereby a slower loading speed further increases the strain at failure obtained [79, p.379]. It is furthermore of common knowledge that the fracture strain increases with increasing concrete strength, however the post-crack behaviour is far more brittle the higher the concrete strength is [79, p.379]. Plastic Concrete is therefore expected to have a higher fracture strain than ordinary concrete and a far more ductile post-peak behaviour. This behaviour has been corroborated by various studies, which identify an achievable strain at failure for Plastic Concrete between 0.5% and 1.0% in unconfined compression tests [2, 28, 99].

Strength Development

Although most reference testing is carried out at 28 days it is of common knowledge in concrete science, that concrete strength continues to increase after 28 days. Concrete curing hereby mainly depends on the cement strength class, cement type and w/c-ratio used [79, p.325f.]. Blastfurnace cement (e.g. CEM III) develops initial strength far slower than ordinary Portland cement (e.g. CEM I), however increases steadily far beyond the 28 day mark [105, p.67f.] [79, p.326f.]. This is due to the latent hydraulic properties of blast furnace slag, which causes a slow but steady strength development. The w/c-ratio also affects hydration rate of concrete, whereby with increasing w/c-ratio the hydration rate decreases [79, p.326]. Furthermore, the cement strength class also influences concrete strength development, with higher cement strength classes causing a more rapid strength development [79, 105]. For strength development of standard concrete, the *fib* Model Code 2010 [88, p.87] gives an approximation for the time function of the concrete strength development as a function of the cement strength class, shown in Equation 3.4.

$$\beta_{cc}(t) = \exp(s \cdot [1 - (28/t)^{0.5}]) \tag{3.4}$$

where	$\beta_{cc}(t)$	:	time-dependant strength development function	(-)
	s	:	coefficient for cement strength class	(-)
	t	:	concrete age	(d)

In line with these considerations, it can therefore expected that Plastic Concrete has a very low hydration rate due to the use of blastfurnace cement, a low cement strength class and a high w/c-ratio. The effect of the cement type and strength has also been shown to be predominant against the bentonite type used [76, p.144f]. Various studies have examined the long-term strength of Plastic Concrete mixtures [2, 22, 28, 89, 110]. In Figure 3.3 an overview of some of these test results is given.

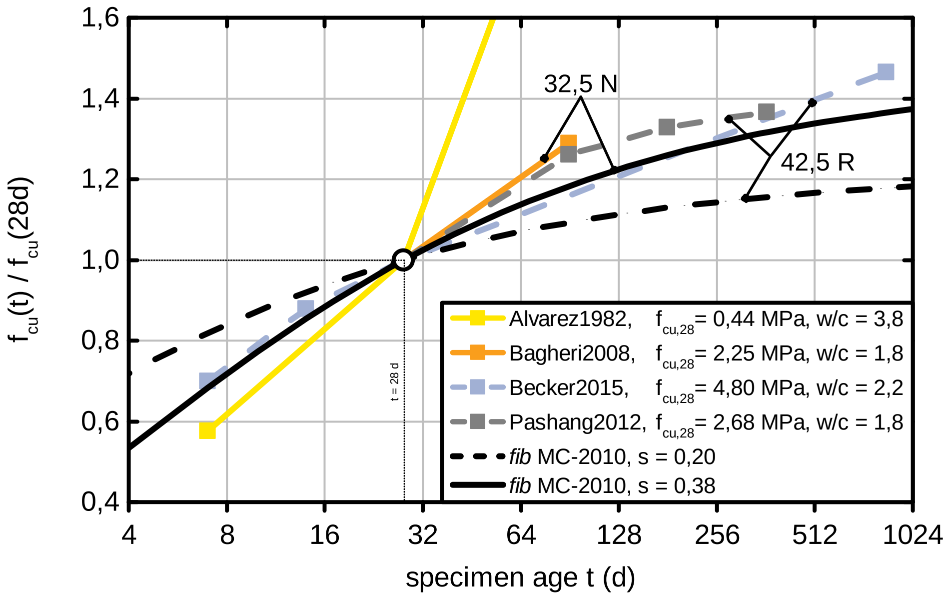


Figure 3.3: Overview of the UCS development as a function of time

As can be seen, the strength development of Plastic Concrete is not finalised after 28 days, instead increasing steadily after 28 days. The studies also show that due to the high w/c-ratio used the strength development of Plastic Concrete, at any given cement strength class, is slower than the *fib* Model Code 2010 estimates. It is also apparent that Plastic Concrete strength increases far beyond the 28 day mark and increases slowly before this date, which strongly relates to the high w/c-ratio used. Alvarez et al.'s tests on Plastic Concrete (with a w/c-ratio of 3.78) for the Convento Viejo Dam project show that even after a 950 day testing (not shown in Figure 3.3), the unconfined compressive strength continues to rise and has not yet reached a plateau [2]. The compressive strength at 950 days is hereby shown to be 5.59 times that at 28 days [2]. Furthermore, studies on cement-treated soils and clays have also shown that the strength increase occurs in a similar manner, with the strength increase being dependant on cement type and cement content used [67, 127]. However, from the literature review, it remains unclear how Plastic Concrete strength development affects the strain at failure of samples, since contradictory results can be found. Some authors provide evidence that with increasing concrete strength (i.e. with sample age) the strain at failure increases [28, 89]. Other authors however ascertain that with increasing concrete strength the failure strain decreases [2, 99, 101]. Against the background of concrete technology it should however be expected, that strain at failure increases with increasing Plastic Concrete strength [79, p.379f.].

The knowledge of the long term strength development of Plastic Concrete is of utmost importance, since cut-off walls are constructed for design periods far exceeding 25 years. It is therefore not essential to test characteristic compressive strengths of Plastic Concrete samples at 28 days and can instead be tested at a higher age. Accordingly, the German standard DIN EN 1538 [47] therefore states, that the knowledge of long-term strength and long-term deformability can be necessary. In addition, the Austrian standard ÖNORM B4452 [108] requires UCS testing to be performed with a sample age of less than 90 days, hereby accepting testing beyond 28 days.

Caution is hereby advised, since a very low strength development may also compromise the construction operation efficiency due to the alternating sequence of primary and secondary panel construction and should therefore be considered during the design phase.

Practical Application

The aforementioned testing was mainly carried out on purposely produced Plastic Concrete specimens under laboratory conditions. In Europe, the concrete samples remain within the formwork for at least 16 h at an ambient temperature of 20 ± 5 °C. Once stripped, concrete samples are placed under water at controlled temperature of 20 ± 2 °C until testing [42]. In Germany, the National Annex requires concrete samples to cure under water for the 6 days subsequent to formwork stripping. After this the samples may be stored in a climate room with temperatures between 15 °C and 22 °C (ideally 20 ± 2 °C) and a relative air humidity of 65 ± 5 % [43]. It is important to take these laboratory conditions into account when evaluating data, since laboratory specimens are cured with negligible confining pressure and have an inexhaustible water supply during curing. On the contrary, tremie-placed Plastic Concrete may encounter differing temperature and humidity conditions. Some authors state for example that a curing temperature of 10 °C or 15 °C is closer to the realistically encountered temperature within the dam body [95, p.239] [55, p.17]. In addition, due to the reservoir water level Plastic Concrete cut-off walls are constantly exposed to water, at least one-sidedly, and should therefore be stored under water until required for testing whenever possible [86, p.N36]. Furthermore, in-situ concrete is subjected to a confining pressure applied by the adjacent ground and the overlying concrete which affects the curing and consolidation of concrete specimens [80, 86].

Additionally, Plastic Concrete cut-off wall integrity is also affected by the tremie method placement, since concrete may displace and destroy part of the filter cake at the ground-bentonite interface in turn causing bentonite or soil inclusion into a single concrete panel as well as water loss. The soil inclusions or water loss may also vary depending on the surrounding soil, since different types of soil have varying stiffness, water content or water permeability [5, 119]. This effect is most commonly noted with cement-treated soils which are produced e.g. with the cutter-soil-mixing (CSM) method, since water loss into the surroundings and water content of soil cause local variations in w/c-ratio [5, 119].

3.2.2 Tensile Strength

Next to the unconfined compressive strength the uniaxial tensile strength (f_{ct}) is an important parameter for the design of concrete structures. For standard concrete the uniaxial tensile strength averages 10% of unconfined compressive strength f_{cu} [105, p.312]. This f_{ct}/f_{cu}-ratio is however not constant and depends on various effects. The ratio decreases with increasing compressive strength f_{cu} [105, p.311]. It furthermore decreases with time, in line with the increase of compressive strength [105, p.311]. Furthermore, the ratio is affected by the type of aggregate, aggregate grading, as well as curing conditions. The f_{ct}/f_{cu}-ratio is hereby higher for wet-cured than air-cured samples. For example *fib* Model Code 2010, suggests that the mean tensile strength can be estimated from the characteristic compressive strength using Equation 3.5 for concrete grades $\leq$ C50 [88, p.77].

$$f_{ctm} = 0.3 \cdot (f_{ck})^{2/3} \tag{3.5}$$

where	f_{ctm}	:	mean tensile strength	(MPa)
	f_{ck}	:	characteristic compressive strength	(MPa)

Following Equation 3.5 for a Plastic Concrete sample with a UCS of 2.0 MPa a mean tensile strength (f_{ctm}) of 0.48 MPa should be expected, suggesting a f_{ct}/f_{cu}-ratio of 0.24. This is in line with the aforesaid deliberations, but differs significantly from the often erroneously implemented f_{ct}/f_{cu}-ratio of 0.10 for standard concrete.

It should be however noted that the mean tensile strength (f_{ctm}) refers to uniaxial conditions, whilst tensile strength testing of concrete specimens most commonly occurs with the splitting tensile strength ($f_{ct,sp}$) test. The European standard DIN EN 1992-1-1 suggests that $f_{ct}/f_{ct,sp}$-ratio can be assumed to be 0.9 [49, p.28]. Recent studies have however shown, that a constant ratio is wrongfully assumed. Malárics' extensive study on standard and high performance concrete shows that $f_{ct}/f_{ct,sp}$-ratio is inversely proportional to concrete strength, with the ratio increasing with decreasing concrete strength [102, p.129ff.], as can be seen in Figure 3.4.

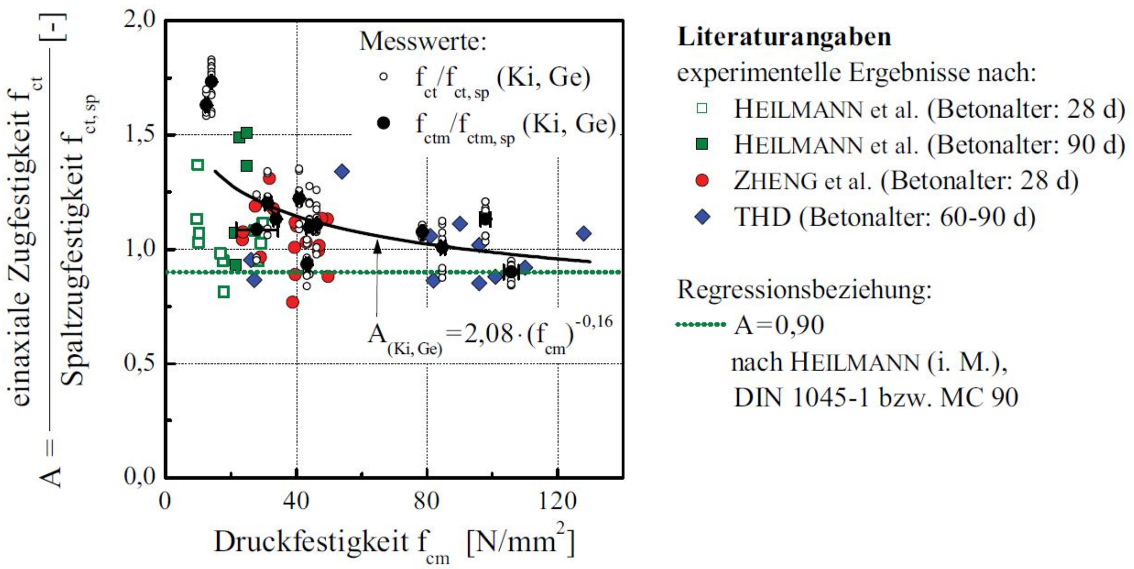


Figure 3.4: Overview of the $f_{ct}/f_{ct,sp}$-ratio of concrete over compressive strength f_{cm} (in German) [102]

The conversion formula presented by Malárics, although limited to strength classes $\geq$ C20, would suggest a $f_{ct}/f_{ct,sp}$-ratio between 1.86 and 2.35 for a UCS of 2.0 MPa depending on specimen shape and aggregate type [102, p.130]. This would imply, that for a $f_{cu} = 2.0$ MPa Plastic Concrete with a tensile strength $f_{ct} = 0.48$ MPa (as estimated with Equation 3.5), a splitting tensile strength of approximately $f_{ct,sp} \approx 1.0$ MPa should be obtained.

The literature review shows that, to date, studies into the tensile strength of Plastic Concrete are scarce. Solely the USACE REMR GT-15 report [91] also tests the splitting tensile strength of concrete. In this study, the authors assumed that the splitting tensile strength ($f_{ctm,sp}$) averages 26% of unconfined shear strength (s) independently of the samples shear strength [91, p.94f.]. With $f_u = 2 \cdot s$ and an estimated $f_{ct}/f_{ct,sp}$-ratio of 2.0 (following [102]) the f_{ct}/f_{cu}-ratio can be estimated to:

$$0.26 = \frac{f_{ctm,sp}}{s} = \frac{f_{ct}/2.0}{f_u/2} \Leftrightarrow \frac{f_{ct}}{f_u} = 0.26$$

This result would be in-line with the results obtained from Equation 3.5, however caution is advised since testing was only limited to on one set of samples with a constant cement content of 300 lb/yd^3 (178.0 kg/m^3) with ages ranging from 3 days to 14 days and does not account for varying bentonite contents and w/c-ratios [91, p.93]. In addition, as mentioned before, the f_{ct}/f_{cu}-ratio is not constant over concrete strength, suggesting possible testing differences, especially since unconfined shear strength was determined with a triaxial testing apparatus. The exact tensile strength to compressive strength relationship f_{ct}/f_{cu} remains however unclear and should therefore be an important part of further investigations.

3.2.3 Multi-Axial Load-Bearing Capacity

Structural concrete is often exposed to multi-axial loading conditions within a structure and the corresponding multi-axial load-bearing capacity of concrete is therefore also of high importance. It should however firstly be remembered that concrete failure under a uni-axial compressive force occurs through the inherent development of a transversal tensile stress and the exceedance of the concrete tensile strength [114, p.305ff.]. The concrete specimen hereby fails through the development of cracks parallel to the direction of main loading exhibiting a brittle behaviour [105]. This lateral strain may however be hindered through the application of a compressive force perpendicular to the direction of main loading, hereby increasing the overall compressive load-bearing capacity of a concrete specimen [114]. By the contrary, if a perpendicular tensile strength is applied, the overall compressive load-bearing capacity decreases. Similarly therefore if a triaxial compression is applied with high lateral stresses, the concrete load-bearing capacity increases manifold [105]. This increase is also known to be more pronounced the lower the UCS and the lower the moisture content of concrete is [79, p.338]. The failure however no longer occurs through the exceedance of tensile strength but instead through crushing, incurring in a change in failure towards ductile behaviour [105, p.295]. Depending on the stress relationship of the three stresses present concrete failure occurs through the development of shear bands and shear failure [79, p.338f.] [114, p.306]. An overview of the failure mode change depending on the stress applied can be seen in Figure 3.5.

The ductile failure hereby occurs at higher strain levels without the presence of a strain-softening behaviour. This change in concrete failure is e.g. reported by Sfer et al. [120], whereby at low confining pressure the failure occurs with propagation of several distributed vertical and inclined cracks. On the contrary, at higher confining pressure the response does not exhibit a well defined peak but a monotonically decreasing slope tending towards a plateau, with failure occuring through sudden propagation of the cracks [120].

In cut-off walls the Plastic Concrete is intrinsically submitted to a multi-axial stress state. However, depending on dam settling and the upstream water level the stress relationship may vary [82]. It is therefore of utmost importance to also understand the multi-axial behaviour of Plastic Concrete. For Plastic Concrete a similar behaviour to standard concrete can be expected. Since the uniaxial com-

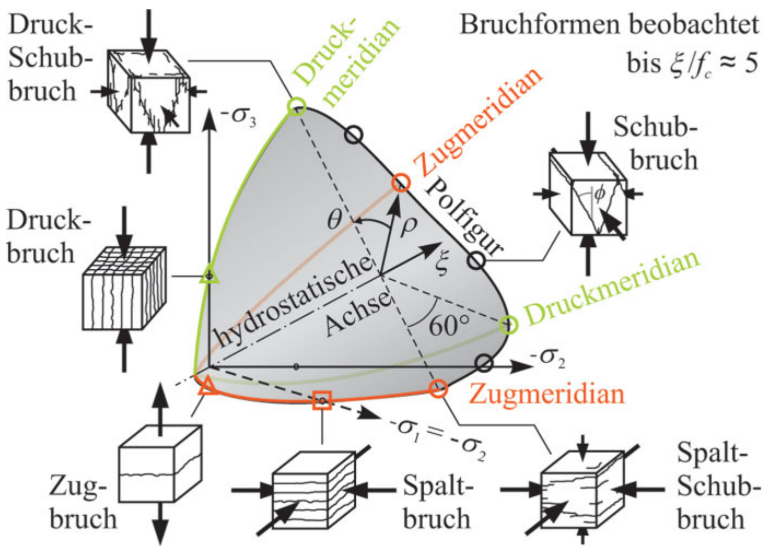


Figure 3.5: Change in concrete failure mode depending on stress level relationship (in German) [121]

pressive strength is low, the multi-axial load-bearing capacity increase can be expected to be more pronounced. However, this increase is likely limited due to the high water and moisture content of Plastic Concrete samples. Various studies have also confirmed the change in failure mode with increasing confining pressure for Plastic Concrete samples [28, 80, 89, 101]. At low confining pressures σ_c cracking occurs parallel to specimen load axis [80], which some authors ascribe to the progressive deterioration of cohesive bonds [101]. At higher confining pressures specimen failure occurs through the existence of failure plane or a mixed failure mode [80], which suggests the dominating frictional properties in specimen failure [28, 101, 110]. The specimens tested at higher confining pressures not only exhibit a higher compressive load-bearing capacity and elastic modulus [89], but also a more ductile, and possibly strain-hardening behaviour and an overall higher strain at failure [28, 89, 101, 110]. An example of this change in behaviour with increasing confining pressure can be seen in Figure 3.6.

Figure 3.6 shows that with increasing confining pressure, both the failure strain and Plastic Concrete strength increase. Various studies have shown, that with confining pressures between 200 kPa and 800 kPa a strain at failure between 2% and 10% can be achieved, emphasising the highly ductile behaviour of Plastic Concrete [28, 89, 101, 110].

Various authors, especially those from the geotechnical area, suggest that Plastic Concrete may be considered a cohesive-frictional material for which a Coulomb-type behaviour can be admitted, similar to very hard soils [101, 110]. Mohr-Coulomb failure criterion states that a material fails if a given combination of normal and shear stresses exceeds the shear strength the material [123]. Accord-

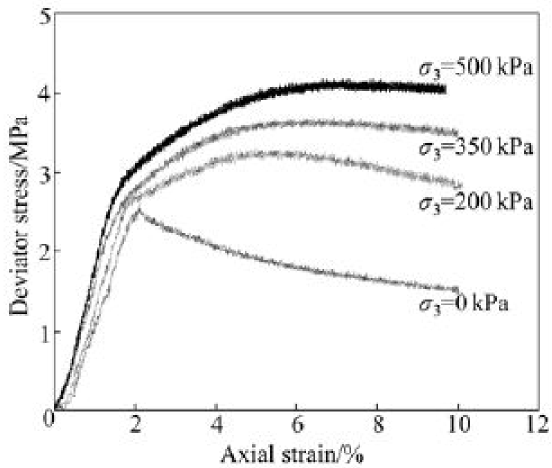


Figure 3.6: Variation of deviator stress versus axial strain for unconfined and triaxial compression tests [110]

ing to the principles of mechanics, the rupture line of the Mohr-Coulomb failure criterion in a three-dimensional environment can therefore be described following Equation 3.6 [117, p.151].

$$\frac{\sigma_1 - \sigma_3}{2} = c \cdot \cos\varphi + \frac{\sigma_1 + \sigma_3}{2} \cdot \sin\varphi \tag{3.6}$$

where	σ_1 :	greatest principle stress	(MPa)
	σ_3 :	smallest principle stress	(MPa)
	c :	apparent cohesion	(MPa)
	φ :	friction angle	(°)

If no radial pressure σ_3 is applied, the Mohr-Coulomb failure criterion given in Equation 3.6 can be simplified to Equation 3.7.

$$\sigma_1 = c \cdot \frac{2 \cdot \cos\varphi}{1 - \sin\varphi} \tag{3.7}$$

where	σ_1 :	greatest principle stress	(MPa)
	c :	apparent cohesion	(MPa)
	φ :	friction angle	(°)

Various studies have however shown that frictional angle φ and apparent cohesion c are not constant for any given Plastic Concrete mix. Hereby cohesion parameter increases and frictional angle decreases with growing specimen age, which the authors ascribe to cement hydration [101, 110]. An increase in cement content further increases the cohesion parameter and decreases the friction angle measured [101, 110]. Inversely, an increase in bentonite content decreases the cohesion parameter and increases the frictional angle of Plastic Concrete samples [110]. Some authors also suggest that a variation in the coarse-to-fine aggregate ratio may change the measured parameters, with an increase in coarse components increasing both apparent cohesion and friction angle [110]. An overview of the effects on mix-design variation on Mohr-Coulomb parameters can be found in Table 3.3.

Table 3.3: Effect of mix-design changes on Mohr-Coulomb parameters

Increase in:	cohesion *c*	friction angle φ
specimen age	+	-
coarse-to-fine-ratio	+	+
bentonite content	-	+
cement factor	+	-

3.3 Elastic Modulus

The elastic modulus *E* of concrete is primarily determined by the elastic moduli of its components cement paste and aggregate, as well as the volumetric proportions of the materials in the mix, and may be estimated through composite theory [114, p.310]. The elastic modulus of cement paste mainly depends on capillar porosity and herewith on the w/c-ratio and degree of hydration. Aggregates generally have a higher elastic modulus which mainly depends on the mineralogical properties of the rock. Therefore, generally speaking, an increase in water content or a decrease in cement content causes the elastic modulus of the obtained concrete to decrease [114, p.310]. It is furthermore common knowledge that with increasing degree of hydration the elastic modulus increases, whereby the elastic modulus increase precedes the compressive strength increase [79, p.298]. This is recognised in the *fib* Model Code 2010 since the time function of the elastic modulus development of concrete $\beta_E(t)$ is estimated to be the square root of concrete strength development $\beta_{cc}(t)$ (see Equation 3.4), as shown in Equation 3.8.

$$\beta_E(t) = [\beta_{cc}(t)]^{0.5} \tag{3.8}$$

where	$\beta_E(t)$:	elastic modulus development function	(-)
	$\beta_{cc}(t)$:	strength development function	(-)
	t :	concrete age	(d)

Plastic Concrete behaves similarly to ordinary concrete. The elastic modulus of Plastic Concrete increases with age and decreases with increasing bentonite content [101,110]. In addition, the elastic modulus decreases with increasing w/c-ratio, in-line with standard concrete behaviour [80, 101, 116, 135]. Some studies have also shown, that the bentonite content also affects the elastic modulus development over time, similarly to the strength development [101, 110]. Furthermore, the elastic modulus further increases with increasing confining pressure during testing in the triaxial testing apparatus [80, 101, 110]. In Figure 3.7 an overview of the test results from various studies is given.

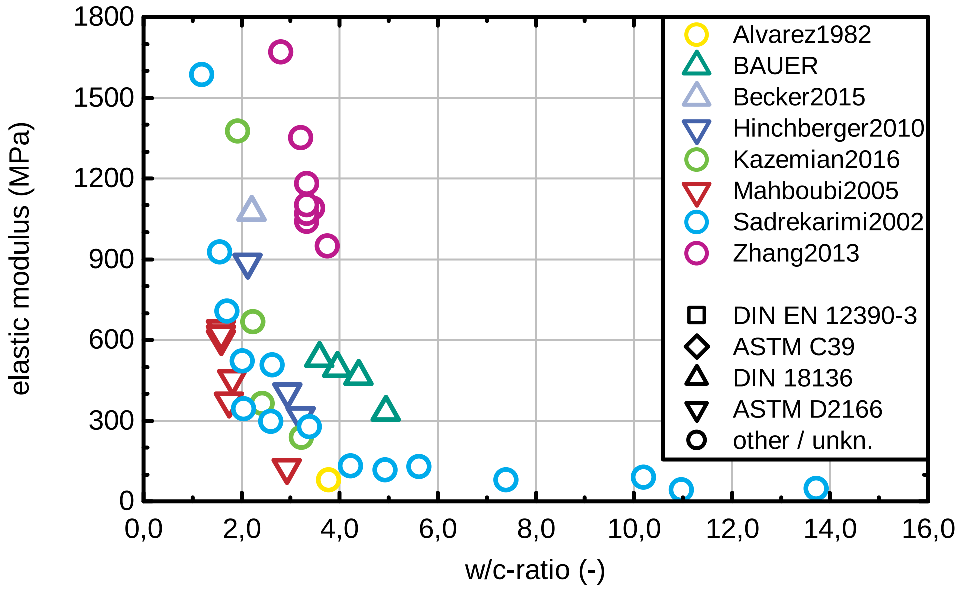


Figure 3.7: Overview of the elastic modulus as a function of the w/c-ratio at 28 days

As can be seen, the elastic modulus generally decreases with increasing w/c-ratio, especially within one data series. However, at one given w/c-ratio the elastic modulus scatters greatly. Taking into account that Zhang et al. [135] tested the specimens following the Chinese concrete standard DL/T5150-2001 [122] it may be suggested that the scatter may be induced by the testing method used. In Figure 3.8 the elastic modulus is plotted over the corresponding compressive strength.

Firstly, the elastic modulus increases with increasing compressive strength. However, it hereby becomes apparent that the testing procedure used clearly influences the obtained elastic modulus, in-line with varying definitions of elastic modulus given in section 3.1. The "elastic modulus" determined with concrete testing standards (Zhang et al. [135]) is higher than that obtained from geotechnical testing standards (e.g. Mahboubi et al. [101]). This is most likely due to the deformation measurement techniques used, since concrete standards measure specimen deformation in-situ (e.g. strain gauges) while geotechnical standards generally use the machine displacement to obtain specimen deformation. Similar results have been found in studies on cement treated soils [104, p.68f.]. This

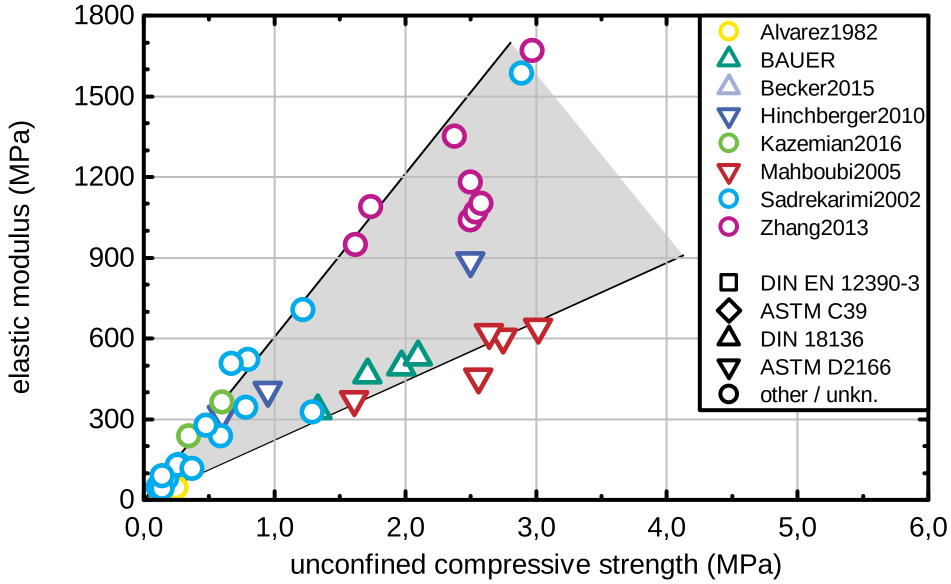


Figure 3.8: Elastic modulus as a function of the compressive strength at 28 days

further substantiates the fact, that the testing conditions should therefore be specified during planning and tendering of projects.

The *fib* Model Code 2010 for example suggests that the elastic modulus of concrete may be estimated with Equation 3.9 [88, p.81].

$$E_{ci} = E_{c0} \cdot \alpha_E \cdot (f_{cm}/10)^{\frac{1}{3}} \tag{3.9}$$

where	E_{ci} :	modulus of elasticity at 28 days	(MPa)
	E_{c0} :	$21.5 \cdot 10^3$	(MPa)
	α_E :	coefficient relating to aggregate type	(-)
	f_{cm} :	mean compressive strength at 28 days	(MPa)

With this, a Plastic Concrete sample with a compressive strength of 2 MPa at 28 days could be estimated to have (with $\alpha_E = 1.0$) an elastic modulus of approximately $E_{ci} \approx 12573$ MPa. It is however clear that, with decreasing compressive strength the elastic modulus decreases disproportionally and therefore can not simply be estimated following Equation 3.9. Based on the literature review, and as shown in Figure 3.7 and Figure 3.8, the elastic modulus of Plastic Concrete can be assumed to be in the range of 300 to 1500 MPa dependant on the testing standard used.

In some cases, when the elastic modulus is determined as the deformation modulus within a triaxial cell, the variation of deformation modulus over confining pressure should be accounted for. It can be expected, that similarly to the compressive load-bearing capacity increase under confining pressure (see subsection 3.2.3), the elastic modulus will also increase with increasing confining pressure. This is in-line with the results shown in Figure 3.9. The results also show that, independently of the sample compressive strength, the elastic modulus increases similarly. However, it should also be noted that some studies also show a decreasing elastic modulus with increasing confining pressure. This phenomena can however only be ascribed to an erroneous measurement of Plastic Concrete specimen deformation.

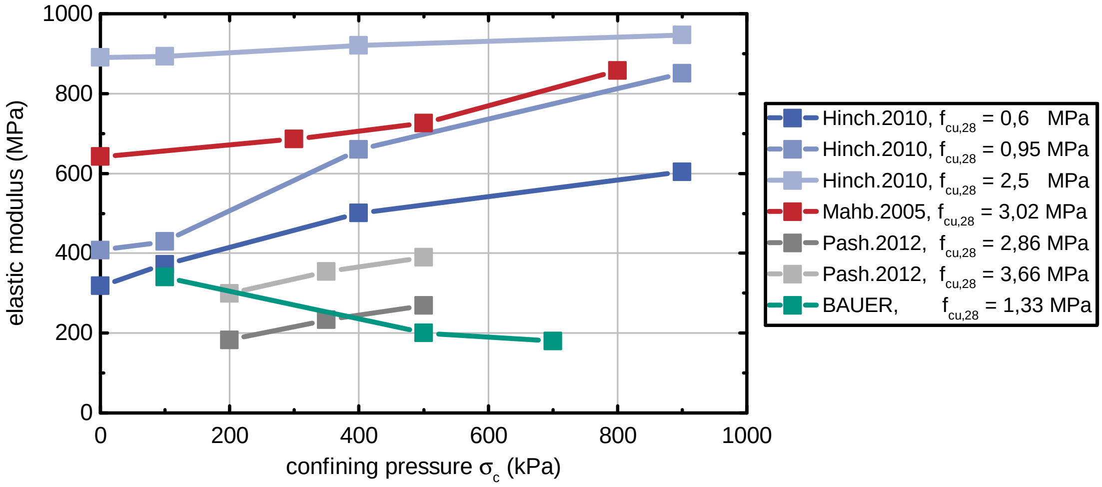


Figure 3.9: Elastic modulus as a function of confining pressure at 28 days

Some studies also investigate how the changes in Plastic Concrete composition may alter the resulting compressive strength [22, 36]. Bagheri et al. for example studies the effect of cement substitution by silica fume [22]. It is especially interesting that with a 15% substitution of cement the Elastic Modulus increases 180% for Plastic Concrete (w/c=1.8), while normal concrete (w/c=0.4) would only increases 30% with the same substitution. As silica fume is known to especially enhance the transition zone quality [105, p.668], the aforementioned 180% increase might suggest, that the transition zone is especially weak in Plastic Concrete mixtures. The authors also state, that the for Plastic Concrete the used of silica fume does not significantly alter the relationship between elastic modulus and compressive strength.

As has been shown, the elastic modulus of concrete directly relates to compressive strength. It is therefore important to note that the requirement of a characteristic compressive strength f_{ck} (defined statistically as the 5-percentile value) is not expedient since this automatically relates to an increase in the elastic modulus [95, p.238]. It is therefore purposive to define a mean compressive strength f_{cm} which is required for the proposed cut-off wall and hereby also establish the targeted elastic modulus.

The literature review has also not presented any results regarding the Poisson's ratio ν_c of Plastic Concrete. For ordinary concrete the Poisson's ratio generally ranges between 0.15 and 0.25, is however mainly dependant on the stress level reaching 0.5 at failure [114, p.311]. Within practical concrete stress levels, the Poisson's ratio of concrete is generally estimated to $\nu_c = 0.20$ [88, 82]. However, due to the high water content in Plastic Concrete samples as well as the presence of clay minerals such as bentonite causing high ductility of Plastic Concrete, the Poisson's ratio is likely to be higher for such samples. As long as Plastic Concrete is still considered an isotropic material, the shear modulus G should still be estimated with Equation 3.10.

$$G = \frac{E}{2 \cdot (1 + \nu)} \tag{3.10}$$

where	G :	shear modulus of concrete	(MPa)
	E :	elastic modulus of concrete	(MPa)
	ν :	Poisson's ratio of concrete	(-)

Some authors have furthermore related the shear modulus *G* measured with bender elements to the shear strength *s* or compressive strength of cement-treated soils [119, 127]. Seng et al. for example suggest a near linear relation $G = 310 \cdot s^{1.06}$ between shear modulus and shear strength [119, p.783]. With a Poisson's ratio $\nu = 0.20$ and $f_{cu} = 2 \cdot s$ [119, p.783] the relation $E \approx 350 \cdot f_{cu}$ can be obtained, which is in-line with results shown in Figure 3.8. However, a perfectly linear relationship is unlikely against the background of concrete technology (as shown in Equation 3.9) and further studies into this relationship should be conducted.

3.4 Creep and Relaxation

When concrete is subjected to a load, concrete firstly reacts elastically. However, besides elastic strain components, concrete also presents a non-linear stress-strain behaviour. When subjected to sustained loading, strain increases gradually with time due to concrete creep. The creep coefficient φ is hereby the most common engineering approach to estimate concrete creep and is defined following Equation 3.11 [88, p.89] [114, p.320].

Various parameters affect the creep behaviour of concrete specimens. With an increasing cement content and increasing water content, concrete creep increases as it is the cement paste phase which undergoes creep [105, p.453]. Normal weight aggregates are generally not sensitive to creep, instead restraining concrete creep. This concrete creep restraint is more pronounced the higher the elastic modulus of the aggregate is [114, p.322]. Some authors also suggest that aggregate grading, maximum size and shape also affect concrete creep [105, p.453]. Furthermore, creep is influenced by the ambient relative humidity, with creep being higher, the lower the surrounding relative humidity [105, p.458] [114, p.322]. In addition, concrete creep increases proportionally to stress within the range of service stresses (normally $\sigma_c < 0.4 \cdot f_{cm}$) [105, p.455] [114, p.320]. Last but not least, concrete creep is also dependant on the age at loading, with creep increasing disproportionally the younger the concrete is at loading [4, p.23]. Therefore, depending on the conditions present the final creep coefficient φ_∞ may vary greatly, normally ranging between $1 < \varphi_\infty < 4$ for standard concrete [114, p.321].

$$\varphi(t, t_0) = \frac{\varepsilon_{cc}(t, t_0)}{\varepsilon_{ci}(t_0)} \tag{3.11}$$

where	$\varphi(t, t_0)$	:	creep coefficient	(-)
	$\varepsilon_{cc}(t, t_0)$	:	concrete creep strain	(-)
	$\varepsilon_{ci}(t_0)$	:	concrete elastic strain	(-)
	t	:	concrete age	(d)
	t_0	:	concrete age at loading	(d)

Furthermore, independently of concrete loading, concrete water loss to its surroundings causes concrete to dry out, in turn causing concrete specimens to shrink. Since the magnitudes of shrinkage and creep are of the same order or greater than those of elastic strain, these must be taken into account in the design process [105, p.413ff.].

On the contrary, if a stressed concrete specimen is subjected to a constant strain, the specimen stress will gradually decrease with time, known as relaxation. Similarly to the creep coefficient, the relaxation coefficient can be used to measure the decrease in specimen stress following Equation 3.12.

$$\psi(t, t_0) = \frac{\Delta\sigma(t, t_0)}{\sigma_0} \tag{3.12}$$

where	$\psi(t, t_0)$	:	relaxation coefficient	(-)
	$\Delta\sigma(t, t_0)$	:	stress decrease	(MPa)
	σ_0	:	initial stress	(MPa)

Both creep and relaxation are based on the same molecular mechanisms and therefore all influences affecting concrete creep also affect concrete relaxation. It is herewith to possible to convert both coefficients into each other following Equation 3.13 [79, p.416].

For a long duration of loading the relaxation parameter is set to $\rho = 0.8$ [114, p.322]. It however remains to be proven, whether Equation 3.13 is also valid for Plastic Concrete.

$$\psi(t, t_0) = \frac{\varphi(t, t_0)}{1 + \rho \cdot \varphi(t, t_0)} \tag{3.13}$$

where $\psi(t, t_0)$: relaxation coefficient (-)
$\varphi(t, t_0)$: creep coefficient (-)
ρ : relaxation parameter (-)

Taking into account the aforementioned influencing parameters, it should be expected that Plastic Concrete has a greater creep and relaxation behaviour than standard concrete. Various studies have confirmed these expectations [30, 80, 99]. Firstly, the very high w/c-ratio will likely incur in high water loss and specimen deformation. Furthermore, the relatively low elastic modulus of some of the components in Plastic Concrete (e.g. bentonite) should further increase the creep strain of Plastic Concrete specimens. In addition, due to the very slow strength development of Plastic Concrete mixtures the specimen loading will likely occur at a low degree of hydration furthering concrete creep. This behaviour is also reported for similarly constituted cement-bentonite and cement-soil mixtures [55, p.31]. Beckhaus et al. for example suggest a final creep coefficient $\varphi_\infty \geq 2$ for Plastic Concrete samples, which they derive from results on soil samples solidified with the jet grouting technique [30, p.217f.]. It can be expected however that Plastic Concrete mixtures may have even higher creep coefficients (e.g. $\varphi_\infty > 3$).

Since concrete shrinkage uninterruptedly occurs, independently of load application and water loss, the experimental set-up for relaxation testing has to continuously adjust the initially applied strain to compensate for concrete shrinkage. Due to these demanding experimental requirements for relaxation testing, not many studies exist relating to concrete relaxation [4, p.21].

Hinchberger et al. [80] studied the effect of constant axial strain on the stress behaviour of Plastic Concrete and found that Plastic Concrete shows significant stress relaxation effects with the measured stress reducing approximately 30% after an 8 h period, as can be seen in Figure 3.10. The authors further note that the stress relaxation process does not stabilise after the 8 h period, suggesting that the long-term stress is likely to be even lower [80]. It should however be noted, that Hinchberger et al. tested their specimens within a geotechnical, triaxial testing apparatus and did not compensate their results with the also inherent concrete shrinkage, as mentioned before. On the other hand some studies have shown that standard concrete has a strength relaxation of 20% after 8 hours [79] as depcited in Figure 3.10. It is therefore necessary to conduct further testing in this area to correctly establish the extent of creep and relaxation present in Plastic Concrete samples. All in all, Plastic Concrete is expected to have a stronger relaxation behaviour than standard concrete and therefore a time-dependant constitutive model is required for Plastic Concrete [80].

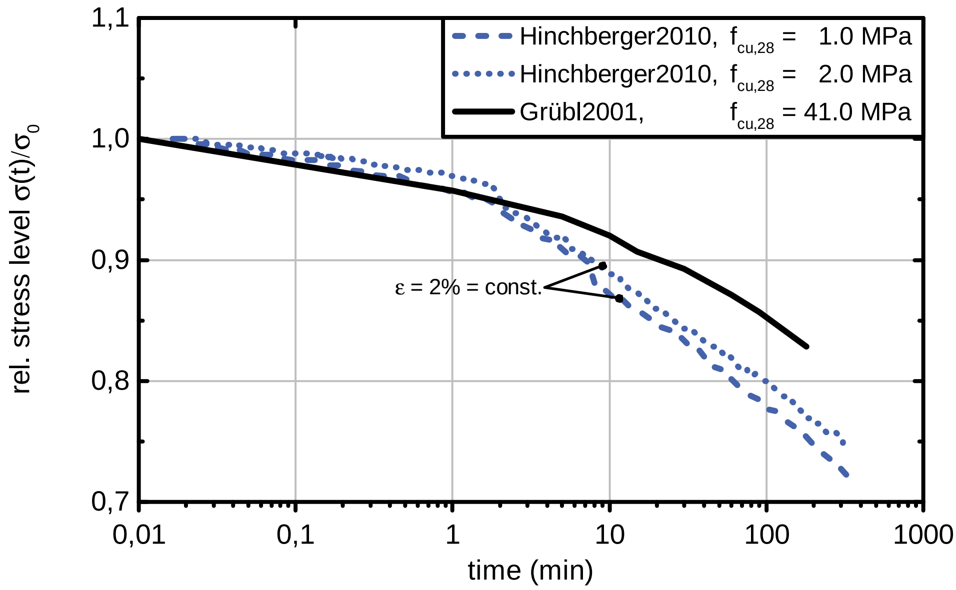


Figure 3.10: Stress relaxation of concrete samples over time

4. Hydraulic Behaviour

The seepage control of earth dams is the main purpose of a cut-off wall, as has been illustrated in chapter 1. Hence, the hydraulic conductivity of Plastic Concrete is one of the most important parameters to be tested. Despite this, no specific testing standard exists for the measurement of Plastic Concrete permeability. Therefore, standard test methods from geotechnical engineering as well as concrete technology are used, which are described in section 4.1. In section 4.2 various studies into the permeability of Plastic Concrete samples are presented.

4.1 Testing Methods

The hydraulic conductivity testing of concrete specimens can be foremost divided into two main testing groups, namely those under loaded and unloaded conditions. Hereby the measurement of hydraulic permeability occurs at simultaneous load or after loading, respectively.

4.1.1 Unloaded Conditions

In concrete technology material hydraulic permeability is normally tested without simultaneous loading. This can occur either on pristine, unloaded samples or on previously loaded samples. The most common testing standard to determine the water tightness of structural concrete samples is DIN EN 12390-8 [46], whereby the depth of penetration of water under pressure is measured. In this, a concrete specimen is placed within a pressure apparatus and submitted to a water pressure 0.5 MPa for 72 h. The specimens are then split and the penetration depth measured. Furthermore, in Germany, air-permeability of concrete is tested following DAfStb booklet 422 [34]. In this, the air permeability of concrete is tested by pressing compressed air through a thin concrete disk in a one-dimensional perfusion state [34].

Most interestingly Hoseini et al. reviewed and analysed many publications regarding the effect of mechanical stress on permeability of concrete, whereby the most of the data reviewed corresponds to testing methods where permeability was measured after crack development [83]. The authors most importantly noted that the permeability of concrete specimens depends on the applied stress, whereby a threshold load level exists. With stresses below the threshold level the permeability of concrete decreases due to the constriction of the pre-existing conduit network. However, above said threshold load level the permeability increases swiftly due to the coalescence of micro-cracks. The authors however note, that the threshold load level is not consistent amongst publications and is furthermore not related to the compressive strength of concrete. Hoseini et al. [83] found that permeability also depends on the crack geometry, whereby a threshold value for crack width in the range of 50 μm to 100 μm exists. The authors also ascertain that due to self-sealing in uncracked concrete and the autogenous healing of cracks, the flow rate decreases with time [83, p.217]. They also state that the concrete mix design affects the permeability of the concrete, as the aggregates are far less permeable than the hydrated cement paste. The authors furthermore note that permeability of concrete is far more dependant on the permeability of its constitutients than on the cement-aggregate interface [83].

Hoseini et al. state that all the tests reviewed vary due to 1) lack of equilibrium in the fluid flow and 2) most of the data is from permeability being measured after cracking. Therefore, future research must account for the effect of thermal cycles on mass transport, as fluid takes days to reach equilibrium. Finally research must focus on a threshold crack width rather than a threshold stress level to determine the onset of critical levels of fluid permeability [83].

4.1.2 Loaded Conditions

In various practical applications (including Plastic Concrete) concrete is submitted to compressive or flexural forces while simultaneously being permeated through. Despite this, the aforementioned testing methods are solely capable of measuring concrete permeability in unloaded conditions. However, a few testing methods exist with which permeability can be measured during loaded conditions for concrete and soil.

Most notably, geotechnical triaxial cells may be used for this purpose. In Germany, the described method is standardised in DIN 18130-1 [58] for laboratory conditions and DIN 18130-2 [59] for field conditions. A sample (most commonly soil) is placed within the triaxial cell and subsequently compressed in axial direction. The specimen is then permeated in axial direction with de-aired water and the resulting flow volume is measured at set intervals. With the given testing geometry, hydraulic gradient i and temperature, the hydraulic conductivity can be determined [118].

Hydraulic conductivity is hereby defined using Darcy's law, which describes the flow of a fluid through a porous medium. This is most commonly used to describe the permeability of soil, whereby an incompressible fluid with constant volume is assumed [117, p.79] and laminar flow within fully saturated samples is implied [118]. In this case, Darcy's law simplifies to Equation 4.1.

$$k_f = \frac{v}{i} \tag{4.1}$$

where	k :	permeability coefficient	(m/s)
	v :	filtration velocity	(m/s)
	i :	hydraulic gradient	(-)

Normally the permeability of soil is determined in triaxial cells with water flowing through sample from bottom to top. The samples have to be fully saturated and therefore water transport through convection can be assumed [118]. Only the capillary pores and the water contained herein are available for convective water transport. The mineral compound can also be considered impermeable [118].

For standard concrete specimens some authors also showed that at low compressive pressures a minor decrease in the measurement of specimen permeability occurs [23]. However, as mentioned before, a threshold value of crack width seems to exist, after which a significant increase in permeability can be expected. In addition, the authors state that the loading history appears to be a critical factor controlling the permeation through stressed concrete. They also note that in their study a decrease in the permeability coefficient occured with time, which the authors ascribe to continued hydration as well as potential pore blocking [23].

It should be noted however that DIN 18130-1 [58] and DIN 18130-2 [59] are both designed for testing granular soils and not for cut-off wall materials or concrete [118]. The main difference between soils and cut-off wall materials (COWM) is their distribution of air voids, since COWM have a larger air void content, which is however filled with water. Soil on the other hand has larger amounts of empty air voids. DIN 18130 [58, 59] states that the de-airing of water should be conducted and the specimen thoroughly saturated [118]. Scholz et al. however showed that, unlike for soils, the deairing of water is not strictly necessary for COWM since the result with non-deaired water are only minimally lower [118]. In addition their study showed no difference between fully-saturated and partially-saturated COWM specimens, which the authors ascribe to the material-intrinsic water-saturation [118].

For concrete specimens other authors have also studied the effect of load level on water permeability [134]. The authors hereby attempt to determine the water permeability of a rectangular concrete specimen by applying flexural load using a centre-placed fulcrum. It is thereby shown that with increasing flexural load the water permeability increases [134]. This study also shows that when the load level exceeds 30% of the crack load f_{cr} the relative permeability increases remarkably. This study should however be considered with caution, as Ø 8 mm reinforcement bars were placed within the concrete specimens, likely affecting the crack formation and also differing from non-reinforced Plastic Concrete behaviour.

4.2 Plastic Concrete Permeability

As mentioned previously, some few studies exist into the hydraulic conductivity of Plastic Concrete Mixtures [2, 22, 28, 72, 80, 101]. It should hereby be noted that, due its low strength, degree of water-tightness and composition, Plastic Concrete is commonly tested following geotechnical testing standards and not structural concrete penetrations tests. An overview of some test results of hydraulic conductivity without confining pressure is given in Figure 4.1.

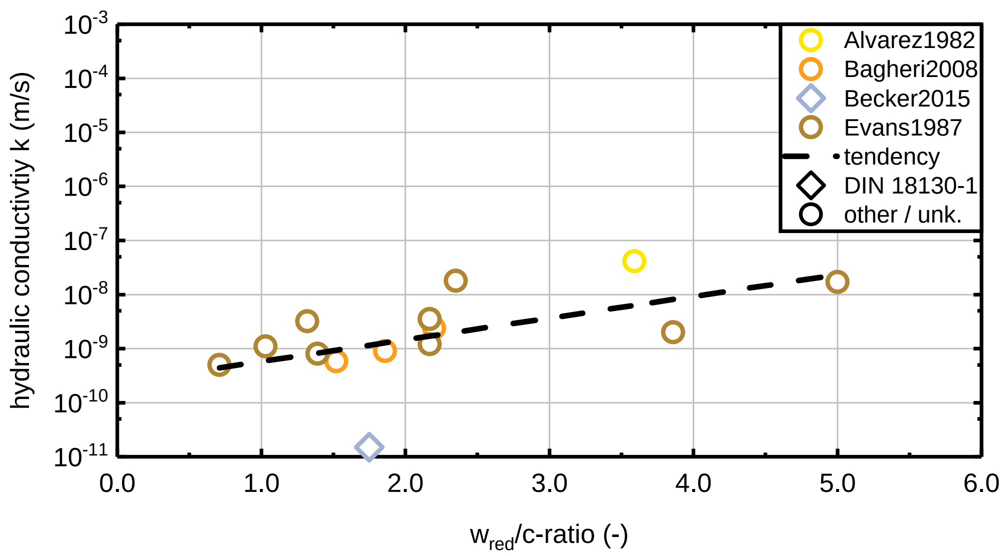


Figure 4.1: Hydraulic conductivity of Plastic Concrete over w_{red}/c-ratio at $\sigma_c = 0$

It can be seen that, similarly to standard concrete, the hydraulic conductivity of Plastic Concrete specimens increases with increasing w_{red}/c-ratio. This may be ascribed to a reduced particle-cross linking and an increased air void content with increasing w/c-ratio. Similarly, when hydraulic conductivity of Plastic Concrete is related to its unconfined compressive strength a decrease in permeability can be observed with increasing compressive strength as shown in Figure 4.2.

The results should however be considered with caution due to the small basis of data available. Plastic Concrete hydraulic conductivity hereby ranges between 10^{-10} m/s and 10^{-7} m/s. Similarly the hydraulic conductivity of Plastic Concrete

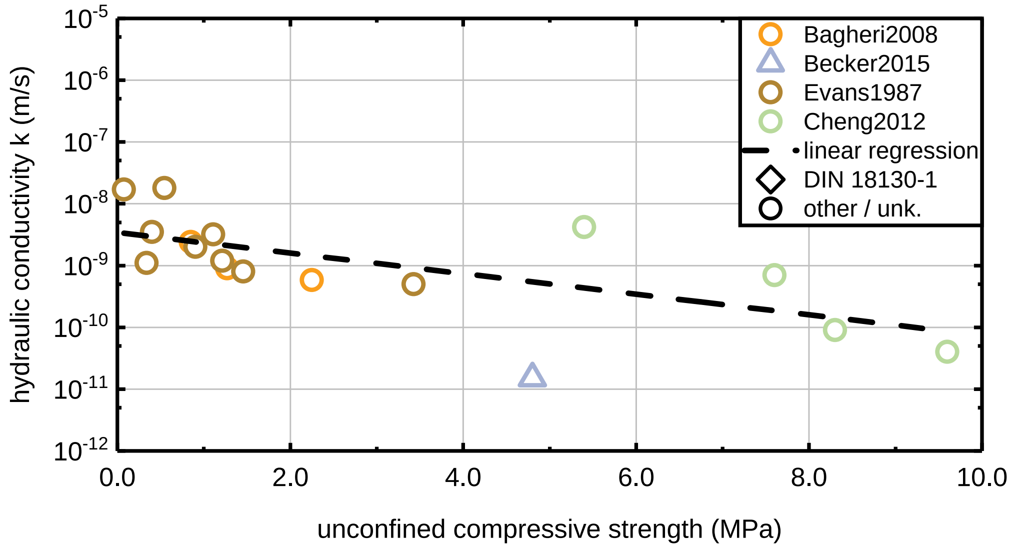


Figure 4.2: Hydraulic conductivity of Plastic Concrete over compressive strength

is consistently lower than that of cement-bentonite diaphragm wall materials [72]. Mahboubi et al. hereby suggest that increasing cement factor decreases permeability, as more water is consumed during cement hydration and less free water remains [101]. Some authors also note that an addition of fly-ash may further reduce permeability, a partial cement substitution by fly-ash on the other hand increases hydraulic conductivity [72]. However, to date, the effect of substitution has only been studied at 14 days, for which reason more detailed and extensive research is necessary. In another study the effect of silica fume on Plastic Concrete hydraulic conductivity was studied [22]. The results showed that through the use of silica fume, and the increase in w/c-ratio to maintain the strength level, a ten-fold reduction in permeability is possible [22]. However, the economic efficiency should be always considered when implementing such substitutions.

Some authors also study the hydraulic conductivity of Plastic Concrete under loaded conditions [80]. Most notably, Hinchberger et al. studied the effect of axial strain on hydraulic conductivity of Plastic Concrete within a triaxial cell. In Figure 4.3 an extract of their results is shown. The authors hereby tested various Plastic Concrete mixtures with distinct compressive strengths at varying confining pressures.

As can be seen in Figure 4.3, hydraulic conductivity increases with increasing axial strain at a given compressive strength and confining pressure. It should be noted that the low strength Plastic Concrete mixtures ($f_{cu} = 0.95$ MPa) follows a relatively smooth curve, whilst the higher strength Plastic Concrete ($f_{cu} = 2.5$ MPa) behaves differently. At low axial strain the stronger concrete, with a possibly more dense structure, exhibits a lower hydraulic conductivity. However, with increasing strain, the hydraulic conductivity increases significantly surpassing that of lower

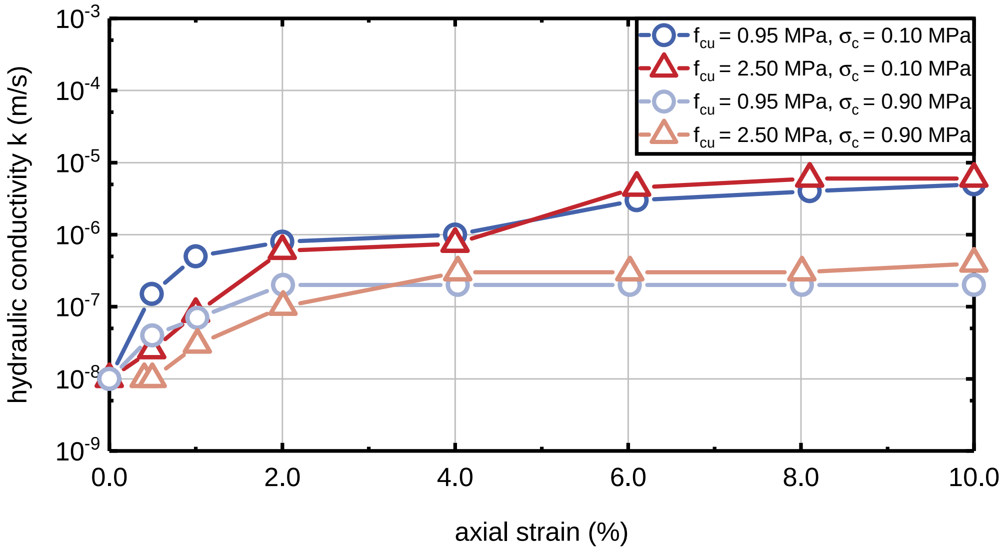


Figure 4.3: Hydraulic conductivity of Plastic Concrete with distinct compressive strengths f_{cu} at varying confining pressures σ_c (following [80])

strength Plastic Concretes at identical axial strains. Similarly to concrete [83], these test results suggest the existence of a threshold strain or threshold stress, whereby the surpassing of this threshold significantly increases hydraulic conductivity.

In addition, the influence of the confining pressure is clearly visible, whereby at higher confining pressure the hydraulic conductivity decreases [80, figs.14-16], which various authors relate to possible pore closing and the pressing together of concrete crack boundaries [80,101]. In all cases however, the combined compression and permeation may cause side-wall leakage during testing which should be accounted for and eliminated through the application of impermeable materials on the lateral surface areas of Plastic Concrete specimens [80].

Furthermore it should be noted that current design procedure for Plastic Concrete does not account for the highly ductile behaviour of this material, whereby a high relaxation and creep potential have been shown to exist (see section 3.4). This behaviour is beneficial for Plastic Concrete hydraulic permeability, since it can prevent material stress peaks during loading and avoid the formation of cracks, which would incur in an increase in permeability. Some initial studies have shown that with deformation of approximately 70% of strain at failure, no significant increase in hydraulic conductivity occurs [99, p.101]. By contrast crack onset in concrete generally occurs at approximately 20% of strain [99].

Finally, some authors suggest that the addition of bentonite reduces the hydraulic conductivity of Plastic Concrete samples [74, 76, 101, 118]. In Figure 4.4 an overview of the hydraulic conductivity over bentonite content is given.

It becomes apparent that the hydraulic conductivity of Plastic Concrete may

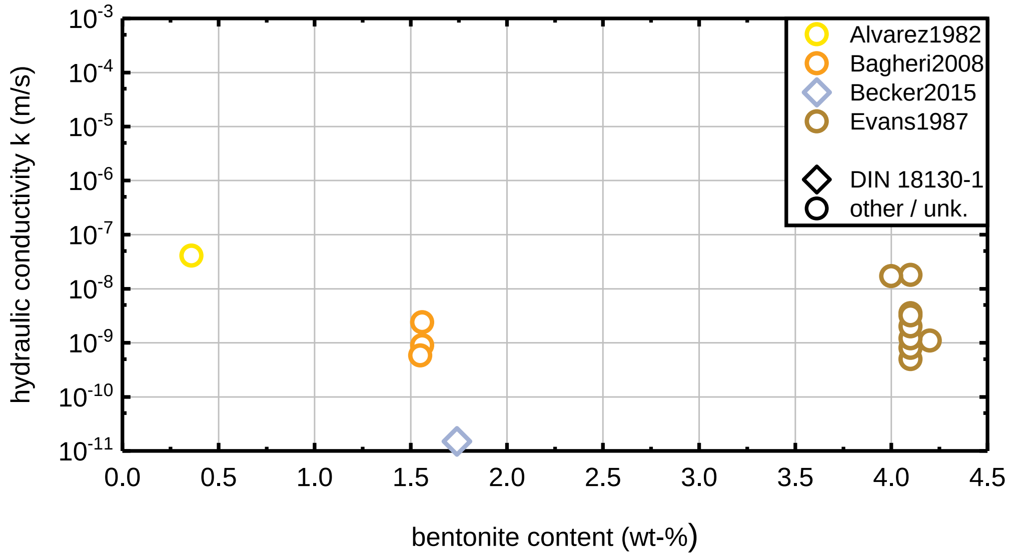


Figure 4.4: Hydraulic conductivity of Plastic Concrete over bentonite content at $\sigma_c = 0$

not be directly related to bentonite content, since the data shows no clear tendency. Some authors further suggest that an optimum specific bentonite content exists, at which the permeability reaches its minimum before increasing again [101]. This can however not be confirmed with the scarce amount of data available in literature and highlights the need for further, systematic research into the hydraulic conductivity of Plastic Concrete.

Only few studies also refer to the time-development of Plastic Concrete hydraulic conductivity [28, 94]. The authors hereby ascertain that with increasing time the hydraulic conductivity decreases [28, 94]. This is in line with the strength development behaviour of Plastic Concrete (see subsection 3.2.1) and is likely caused by the progress of hydration and the formation of CSH phases consolidating cement particles together [84]. In addition, crack self-healing and crack obstruction with the transported particles, amongst others, are also known to cause the permeability of concrete to further decrease over time [69, p.35].

However, some authors suggest that depending on Plastic Concrete mix design and materials used, the hydraulic conductivity may increase slightly at higher age [28], which may be related to mechanical and chemical actions on Plastic Concrete cut-off walls. Some specifications therefore allow for permeability tests at higher ages (e.g. 90 days), to achieve the required design permeability values [86, p.N13]. The Austrian standard OENORM B 4452 for example allows Plastic Concrete permeability testing to be performed up to 90 days of age [108, p.9]. It may however also be contractually expedient to set 28-day control values, not as the design permeability but as a demonstration of design value achievement, to shorten the acceptance period of the construction services provided [86, p.N13].

5. Summary and Outlook

5.1 Summary

With the present report first steps are set out for a comprehensive understanding of Plastic Concrete material behaviour. It can be concluded that Plastic Concrete is a low strength concrete due to its material behaviour, mix design and placement method. With the acquired knowledge Plastic Concrete can be used to safely guarantee seepage control inside and below dams with a controlled material behaviour. All in all, the following considerations may be taken into account for Plastic Concrete cut-off wall design.

5.1.1 Mix Design

Plastic Concrete can be considered to be a low strength concrete with a low elastic modulus capable of sustaining larger strains than normal concrete. These properties can be achieved through the targeted selection of source materials and mix design. Equally to standard concrete, cement is used as a binder in Plastic Concrete. Although ordinary Portland cement is most commonly used, blast-furnace or pozzolanic cement may also be used. These types of cement are known to incur in slower concrete strength development and lower permeability of the hardened material. The key component differentiating Plastic Concrete from ordinary concrete is the far higher w/c-ratio, for which the fresh concrete stability has to be controlled by low amounts of physically water-binding additions. Most commonly bentonite, a clay-rock composed of montmorillonite minerals, is added since bentonite is known to adsorb a great amount of water and hence is implemented as a stabilising agent. To date, bentonite characterisation is however only limited to performance-based testing procedures. Possible water-binding additions are not however limited to bentonite. Finally, Plastic Concrete uses regular aggregate for its mix design with the maximum aggregate size being often limited to 16 mm due to the segregation risk with greater maximum grain size. As admixtures retarding agents are commonly implemented to delay concrete setting in tremie placement, whilst super-plasticizing admixtures are sometimes used to increase concrete workability.

Plastic Concrete mix design is similar to that of standard concrete. The aggregate content ranges from 1300 to 1900 kg/m^3. Cement content is normally in the range of 80 to 200 kg/m^3. The w/c-ratio generally ranges between 2.0 and 5.0, however the exact value depends mostly on the target strength and source materials used. The mixing sequence also affects material behaviour, whereby currently no common mixing sequence exists. Usually, bentonite and water are premixed to a bentonite slurry. The bentonite slurry is then allowed to hydrate for a period of time, between 3 hours and 24 hours. After this, the bentonite slurry is mixed with cement, sand and gravel. Alternative mixing procedures do however exist and there influence on the final material properties should be the aim of future studies.

5.1.2 Mechanical Behaviour

The mechanical behaviour of Plastic Concrete is in line with that which can be expected from concrete technology. It should however be noted that much testing is conducted using geotechnical testing standards and not concrete testing standards. This difference is especially important when testing the elastic modulus of Plastic Concrete to asses the materials' deformability.

Generally speaking, it can be ascertained that the compressive strength of Plastic Concrete increases with decreasing w/c-ratio. It should be noted however that the w/c-ratio does not account for the addition of bentonite and therefore not consider the reduction in free water available for cement hydration. Plastic Concrete compressive strength normally lies between 0.5 to 2.5 MPa at 28 days. In addition to this, it is known that the compressive strength development is very pronounced for Plastic Concrete, far beyond the 28 day mark. The magnitude of strength development clearly depends on the cement type used (ordinary Portland, blast-furnace or pozzolan cement). It may therefore also be expedient to test Plastic Concrete compressive strength at higher ages, e.g. 90 days. Furthermore, for the purpose of simplification, it is also recommended to establish a minimum compressive strength of at least 0.3 MPa at 28 days to ensure erosion stability of Plastic Concrete mixtures.

The strain at failure of Plastic Concrete is also far greater than that of standard concrete, where under compression a maximum strain of up to 1% can be achieved. It may therefore be expedient to establish a maximum strain at failure in the range of 0.8% to 1.0% as a requirement. It should however be noted that concrete behaves softer (i.e. has a lower Young's modulus) the lower the compressive strength is.

The tensile to compressive strength ratio of Plastic Concrete is also expected to be greater than that of standard concrete, with the tensile strength being in the range of 10% to 20% of compressive strength. Under multi-axial load, the load bearing capacity clearly increases with axial strains as high as 10%.

The magnitude of the elastic modulus of Plastic Concrete clearly depends on the testing standard used. On the one hand the definition of elastic modulus is different for concrete standards (i.e. defined as the secant modulus within a testing procedure) and geotechnical standard (i.e. defined as the maximum tangential modulus in unconfined compression testing). The deformation is also measured inconsistently with the geotechnical standards commonly measuring piston movement or occasionally measuring loading plate movement (ex-situ) whilst concrete standards require deformation measurement of the sample through strain gauges, LVDTs or similar (in-situ). The deformation modulus (geotechnical standard) of Plastic Concrete can therefore be estimated to 100-600 MPa, whilst Young's modulus (concrete standard) should be estimated in the range of 300-1800 MPa. In addition, as would be expected, the elastic modulus increases with increasing confining pressure.

Since Plastic Concrete is generally mixed with a high w/c-ratio, the creep and relaxation properties are more pronounced than those of standard concrete. With this the final creep coefficient can be expected to be $\varphi_\infty \geq 3.0$. Therefore the relaxation potential of Plastic Concrete is also notably higher than that of standard concrete, which is however not accounted for in Plastic Concrete design. The higher relaxation potential of Plastic Concrete is in turn beneficial to prevent material stress peaks during loading and avoid the formation of cracks, which would incur in an increase in permeability. It should be underlined that, to date, the relaxation behaviour is not taken into consideration in material design.

5.1.3 Hydraulic Behaviour

The hydraulic behaviour of Plastic Concrete, and concrete in general, remains a relatively unstudied field, especially for testing under realistic stress conditions. The testing of hydraulic conductivity is hereby not standardised for concrete, whereby two groups of testing conditions can be differentiated, namely testing under loaded and unloaded conditions. Most commonly testing is conducted under unloaded conditions, whereby concrete specimens have sometimes been preloaded prior to testing. Various studies have shown that concrete permeability mainly depends on a threshold crack value, not a threshold stress value, after which permeability increases significantly.

For Plastic Concrete it has been shown that permeability decreases with decreasing w/c-ratio which is linked to a less porous material structure. The literature review has also shown that an increase in bentonite quantity alone can not be related to a decrease in Plastic Concrete permeability.

In a triaxial cell, permeability testing can be conducted with simultaneous triaxial compression. When testing under these conditions it can be seen that a threshold value exists after which permeability of Plastic Concrete changes. High strength Plastic Concrete (> 2.5 MPa) has a lower initial permeability however under increased strain, permeability increases far more significantly than for lower strength Plastic Concrete. These results should however be considered with caution, since permeability testing in triaxial cells is designed for soil testing and has not been studied in detail for concrete specimens. Furthermore, as would be expected, an increase in confining pressure has shown to reduce Plastic Concrete permeability.

The change in Plastic Concrete permeability over time is scarcely reported in literature, however a decrease in permeability over time has been shown to exist. This is in line with concrete technology where with progressive cement hydration the permeability decreases through the increasing cross-linking of particles. This is especially relevant with blast-furnace or pozzolan cements where increased cross-linking occurs at higher ages (pozzolan effect). It is therefore expedient that Plastic Concrete permeability testing is conducted at ages greater than 28 days (e.g. 90 days) to account for the permeability increase with time. This in turn represents the long-term material behaviour of Plastic Concrete more realistically. Plastic Concrete permeability can therefore be estimated in the range of $1 \cdot 10^{-8}$ m/s to $1 \cdot 10^{-9}$ m/s depending on testing age.

5.2 Future Research

With the present report the State-of-the-Art for Plastic Concrete in cut-off walls has been established. Despite these promising results, questions remain which should be the purpose of further studies.

Firstly, further research is required to examine the effects of the mixing procedure on Plastic Concrete hardened behaviour. The focus of these studies should be placed at understanding the interaction of water, bentonite and cement and to what extent the varying mixing procedures may alter the availability of water during cement hydration. Further studies into the mix design and the source materials used in Plastic Concrete may also identify optimisation possibilities for Plastic Concrete material behaviour. Most notably reliable analytical methods must be studied to comprehensively characterise bentonite source materials as this may shed light on the mechanism underlying Plastic Concrete behaviour and establish bentonite requirements. The understanding of these mechanisms is also of utmost importance to establish their influence on compressive and tensile strength of Plastic Concrete as well as creep behaviour.

Furthermore, research is needed to determine to what extent bentonite affects the final material behaviour and establish the effectiveness of bentonite to increase Plastic Concrete ductility and deformability. In addition, the long-term behaviour of Plastic Concrete should be further studied to determine permeability and compressive strength change over time.

Of transcendent importance is the need to further investigate the creep and relaxation potential of Plastic Concrete, since these have a significant impact on the material stress. This in turn strongly affects cut-off wall design since a high relaxation potential provides a far greater potential deformability before incuring in cut-off wall damage. In addition, further research should be undertaken to confirm the increased tensile to compressive strength ratio present for Plastic Concrete. Finally, a comparative, experimental study between concrete and geotechnical testing standards is of high interest to enable the transposition of test results into one another and determine the causes for the disparate test results.

On the other hand, the permeability changes in Plastic Concrete should be the subject of further studies. A greater focus on the determination of Plastic Concrete permeability under simultaneous loading could produce important findings that account for a more realistic design of Plastic Concrete cut-off walls. For this, the development of a new testing method may also be necessary. Further studies are also need to be carried out to validate the reported decrease in Plastic Concrete permeability over time, whereby a distinction of the source materials used should be made. It would also be interesting to assess the effects of different bentonites on the Plastic Concrete permeability values and relate these to bentonite structure characteristics. Last but not least, further studies into the identification of threshold strain value for Plastic Concrete permeability increase must be conducted to safely design cut-off walls and ensure their imperviousness to water during operation.

All in all it may be summarised that the findings of this study have a number of important implications for future practice. However, continued efforts are needed to further understand Plastic Concrete behaviour and ensure its correct application in cut-off wall design.

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