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Technical Note

Review of SKB's continued work on design development for the 2BMA disposal vault of the SFR repository

2024:15

Author: Jon Knights, Richard Metcalfe
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SSM perspective

Background
The Swedish Radiation Safety Authority (SSM) examines the Swedish Nuclear Fuel Company's (SKB) applications in a step-wise review and approval process according to the government's licence conditions under the Act on Nuclear Activities (SFS 1984:3) for the construction and operation of geological disposal facilities. As part of the review, SSM commissions consultants to carry out work in order to obtain information on specific issues. The results from the consultants' tasks are reported in SSM's Technical Note series.

Objectives of the project
The objective of this project was to support SSM in its consideration of issues linked to the design, construction and long-term structural integrity of the 2BMA disposal vault in the planned expansion of the SFR facility for the final disposal of short-lived low- and intermediate-level waste. The review covers SKB's choice of methods and materials as well as the results of large-scale tests carried out by SKB regarding the feasibility of building the concrete caissons. The assignment also includes an assessment of the suitability of the models and data used by SKB to represent long-term degradation of concrete.

Summary by the author(s)
An in-depth review of the work carried out by SKB on the 2BMA disposal vault design has been carried out. Whereas the range of work undertaken is judged to be impressive, and its reporting logically structured, the development of concrete composition and caisson design from that presented as a basis for SKB's license application could be more clearly presented. It is noted that SKB acknowledges some practical challenges associated with caisson construction using the proposed methods and materials, which indicate that there is scope for further optimisation prior to installation, in order to ensure that the required consistency of quality is achieved. Suggestions are made for potential improvements regarding the concrete mix design and construction methods. It is important that SKB demonstrates understanding of the impact of any changes in concrete composition in the final design on future evolution of the 2BMA caissons and their possible degradation, from the perspective of fulfilling barrier safety functions.

Project information
Contact person at SSM: Michael Egan

SSM perspektiv

Bakgrund

Strålsäkerhetsmyndigheten (SSM) granskar Svensk Kärnbränslehantering AB:s (SKB) ansökningar i en stegvis prövnings- och godkännandeprocess enligt regeringens tillståndsvillkor enligt lagen (1984:3) om kärnteknisk verksamhet avseende uppförande, innehav och drift av geologiska slutförvarsanläggningar. Som en del i granskningen ger SSM konsulter uppdrag för att inhämta information i avgränsade frågor. I SSM:s Technical note-serie rapporteras resultaten från dessa konsultuppdrag.

Projektets syfte

Syftet med detta projekt var att stödja SSM i sin övervägande av frågeställningar kopplade till utformning, uppförande och långsiktig strukturell integritet av förvarsdelen 2BMA i den planerade utbyggnaden av SFR-anläggningen för slutförvaring av kortlivade låg- och medelaktivt avfall. Granskningen omfattar SKB:s val av metoder och material samt resultat av storskaliga tester utförda av SKB avseende genomförbarheten av att bygga betongkassunerna. I uppdraget ingår även en bedömning av lämpligheten av de modeller och data som SKB använder för att representera långsiktig degradering av betong.

Författarsammanfattning

En fördjupad genomgång av SKB:s arbete med utformningen av förvarsdelen 2BMA har genomförts. Medan omfattningen av utförda arbeten bedöms vara imponerande och rapporteringen logiskt uppbyggd, skulle utvecklingen av betongsammansättning och kassunutformning från den som presenterades som underlag för SKB:s tillståndsansökan kunna presenteras tydligare. Det noteras att SKB identifierar några praktiska utmaningar i samband med byggandet av kassunerna enligt de föreslagna metoderna och materialen, vilka indikerar att det finns utrymme för ytterligare optimering inför uppförande, för att säkerställa att erforderlig fasthet i kvalitet uppnås. Förslag lämnas på potentiella förbättringar avseende betongblandningens utformning och konstruktionsmetoder. Det är viktigt att SKB visar förståelse för inverkan av eventuella förändringar i betongsammansättningen i den slutliga designen på framtida utveckling av 2BMA-kassunerna och deras eventuella degradering, ur perspektivet att uppfylla kraven på barriärens säkerhetsfunktioner.

Projektinformation

Kontaktperson på SSM: Michael Egan
Diarienummer avrop: SSM2022-2334
Aktivitetsnummer: 5520028

Contents

1. Introduction

This report is an in-depth review of SKB’s continued work on the design of the 2BMA disposal vault of the SFR repository for short-lived radioactive waste. The SFR has been in operation since 1988 and the 2BMA vault is planned to be a component of an extension to the facility, termed SFR3. It is intended that the 2BMA vault will receive intermediate-level radioactive waste (ILW).

SKB applied for a licence to extend the SFR in 2014 and a licence was subsequently issued to SKB in accordance with the Nuclear Activities Act in Sweden. However, to proceed further SKB also needs a licence in accordance with Swedish Environmental Code, which requires approval from the Land and Environmental Court. The approval was granted in November 2022 and enabled SKB to commence a safety assessment which is the next step in the process to obtain a construction licence for the extension to the SFR. This assessment was submitted to SSM at the end of March 2023. The review presented in this report was carried out as a contribution to SSM’s preparation for reviewing this safety assessment and was undertaken between 23rd November 2022 and 28th February 2023.

During SSM’s regulatory review of SKB’s licence application for the expansion of the SFR facility, SKB updated plans for the design of the 2BMA vault as a result of questions raised by the Authority (SKB, 2017). The review has therefore covered several reports that describe this updated design that have been published by SKB since the submission of the licence application in 2014. Account is taken of additional external references identified by the reviewers that are related to the review topic as defined by SSM. The review:

- assesses and documents the overall quality of SKB's documentation listed below;
- includes an account of the structure and most relevant parts of SKB's documentation;
- assesses the progress made in the area with regard to SSM's expectations from its review of the initial safety analysis report (F-PSAR) submitted with the licence application;
- assesses the transparency, traceability, scientific soundness, and maturity of SKB's technical solutions and of SKB's methodology;
- where applicable, assesses the adequacy of relevant models, data and derived safety functions, as well as the approach to handling of uncertainties; and
- states the merits and weaknesses of SKB's work.

Hence, an assessment is made about:

- The reported efforts by SKB regarding developments in design and construction, e.g. choice of methods and materials as well as the feasibility of construction, for the layout of the disposal vault 2BMA.

- The degree of agreement between estimated safety critical properties of the concrete construction, e.g. hydraulic conductivity and porosity, as assumed in SKB's safety analysis and results obtained from the installation of prototype caissons at the Äspö Hard Rock Laboratory.

The primary documents specified by SSM are:

- SKB TR-18-12. Large scale test of design and material. Svensk Kärnbränslehantering AB (Mårtensson and Vogt, 2019).
- SKB TR-20-09. Concrete caissons for 2BMA. Large scale test of design, material and construction method. Svensk Kärnbränslehantering AB (Mårtensson and Vogt, 2020).

Other reports specified by SSM for review are:

- Further developed design of the repository space 2BMA in expanded part of SFR. Report 1569813, 2017 (translated) (SKB, 2017).
- SKB R-13-40. The impact of concrete degradation on the BMA barrier functions. Svensk Kärnbränslehantering AB (Höglund, 2014).
- Development of structural concrete for the caissons of 2BMA. 2017. SKB R-17-21 (translated) (Lagerblad et al., 2017).

To gain understanding of these reports, additional documents were consulted and are referenced in the text below.

The scope of the study is divided into the following chapters:

- Section 2 provides a brief description of the SFR facility and 2BMA vault.
- Section 3 provides review summaries of the main documents for the 2BMA construction trials, SKB TR-18-12 and SKB TR-20-09.
- Section 4 discusses the main points of concern within the review, focussing on the most pertinent points.
- Section 5 reviews SKB's work on concrete degradation.
- Section 6 provides summary conclusions.

2. Project Appreciation

2.1. SFR Repository Description

The repository for short-lived radioactive waste, SFR has been in operation since 1988 and hosts operational waste from the Swedish nuclear power plants and from the other nuclear facilities. The repository includes underground waste vaults along with surface technical facilities. The waste vaults are located in granitic bedrock at a depth of about 60 meters below the Baltic Sea and are reached via two one-kilometre-long access tunnels from the ground surface (SKB, 2015; Mårtensson and Vogt, 2019, 2020).

The underground part of the existing facility, named SFR 1, consists of four 160-metre-long waste vaults and one 70-metre-high silo (right of Figure 1). It is planned to increase the storage capacity of SFR through the construction of six new waste vaults, which will be located at a depth of 120 metres, i.e. at the same level as the bottom of the silo. These six waste vaults which are shown in the lower left part of Figure 1 will comprise four vaults for low-level waste (2–5BLA), one vault for intermediate level waste (2BMA) and one vault for reactor pressure vessels (BRT) each with a length of up to 275 meters.

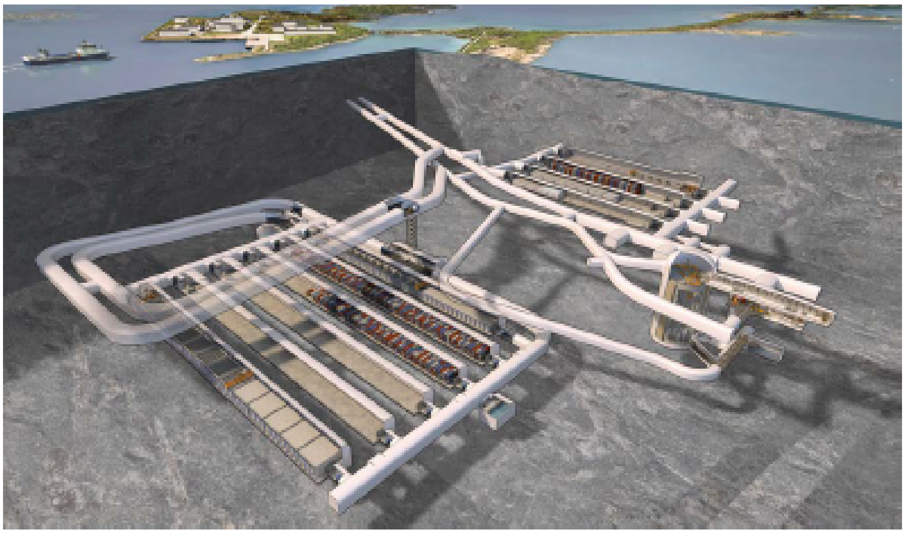

Figure 1: Illustration of SFR. The existing facility is shown in the top right and part of the planned extension (2BMA) in the lower left (SKB TR-18-12).

2.2. 2BMA Description

The 2BMA vault will be around 275 m in length, which will comprise a number of concrete caissons that contain the waste packages for disposal.

The dimensions of the caissons are altered through the reports. The reference design also currently assumes unreinforced concrete.

Figure **2** illustrates the operational phase of the 2BMA vault.

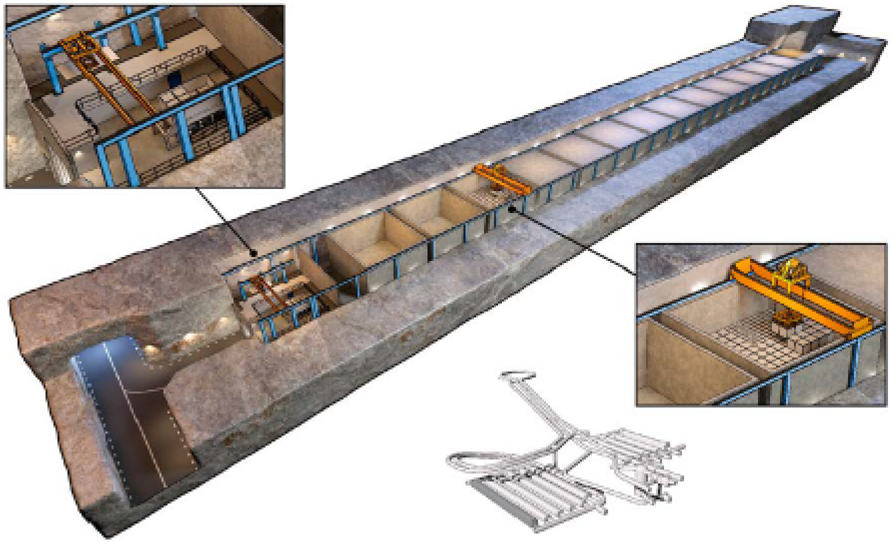

Figure 2: Illustration 2BMA during the operational phase. Inserted details show emplacement of waste packages (SKB TR-18-12).

Maximum permissible crack width according to the reference design is 0.1 mm.

2.3. Environment of the 2BMA Vault

Although the SFR is currently located below the Baltic Sea the site is uplifting (SKB, 2013), and as a consequence of the uplift it will emerge above sea level at around 3000 years in the future. The position of the shore will affect the flow and chemistry of groundwater around the 2BMA vault. Beyond 3000 years, in the Main Scenario of the SFR-PSU safety assessment, the area of the SFR remains above sea level and conditions fluctuate between periglacial and temperate until the end of the assessment period at 100,000 years (SKB, 2015). After about 50,000 years in the future in this scenario about 50% of the time conditions are periglacial and about 50% of the time temperate. Prior to 50,000 years conditions are mostly temperate. However, the actual future evolution of the environment affecting the SFR is uncertain and will be affected by anthropogenic global warming, the severity of which will depend partly on future human actions.

During the Quaternary period (2.6 Ma to present) the area around the SFR has been subjected to repeated cycles of glaciation and deglaciation, accompanied by isostatic depression and uplift with consequent changes in sea level (SKB, 2013). These variations have been accompanied by changes in the groundwater chemistry of the area at the depths of the SFR. In temperate periods when the area has been above sea level meteoric water has recharged. When the area has been covered by ice, fresh sub-glacial water has recharged, or recharge has been cut off, depending on the temperature conditions at the base of the ice. There have also been periods of periglacial conditions when ground freezing has restricted recharge. Saline water has recharged when the area has been below sea level. The groundwater presently at the depth of the SFR extension is a mixture of water with all these origins. There is no evidence for fully marine conditions; present Baltic seawater is substantially less saline than open ocean water such as that in the Atlantic (present Baltic seawater has total dissolved solid content (TDS) of about one sixth of that of open marine water).

Presently, natural groundwater around the SFR has Na-Cl and Ca-Na-Cl dominated chemistry with Cl content ranging between about 1,600 and 5,500 mg/L (SKB, 2013). However, the groundwater system has been disturbed by the SFR, with Baltic seawater being drawn downwards. In the undisturbed system, the most saline groundwater occurs at depths of 100 m-200 m and is of the brackish marine Na-Cl dominated Littorina type; this is the water that will probably occur around the 2BMA vault post-closure following recovery of the hydrogeological disturbance. The Littorina sea can be thought of as a proto-Baltic Sea, the salinity of which was a maximum between about 4500 BCE and 3000 BCE (Laaksaharju et al., 2008). The more dilute brackish water found at shallow depths (~ 100 m) is of local Baltic type, and the most dilute waters (1,600 mg/L Cl) are a mixture of brackish and glacial water and occur at c. 240 m depth.

2.4. Safety Functions

The safety functions of the 2BMA concrete caissons are described in SKB (2014, 2015). For the SFR overall, there are two primary safety principles:

- Limitation of the activity of long-lived radionuclides in the waste.
- Retention of radionuclides.

The 2BMA concrete caissons contribute to meeting the second of these safety principles by:

- helping to ensure low water flows in the vault; and
- ensuring good retention of radionuclides within the concrete, principally by sorption.

The low water flows are achieved by the hydraulic contrast between permeable macadam backfill surrounding the concrete structures in the 2BMA vault and the less permeable concrete (SKB, 2014, 2015). This contrast diverts water flow away from the concrete structures to the more permeable surrounding materials (i.e. provides a hydraulic cage). This is the primary safety function of the caissons.

The relevant safety function indicators related to good retention are:

- pH in concrete barriers (alkaline, initially pH 13);
- redox potential in concrete barriers (reducing);
- concentrations of complexing agents in concrete barriers (which should be low to prevent or limit the formation of complexes with radionuclides); and
- available specific surface area for sorption in concrete barriers (with increased specific surface area favouring sorption).

These safety functions are of secondary importance for the caissons, being provided mainly by cementitious grout surrounding the wastes.

According to SKB (2014), the 2BMA vault will be constructed in such a way that generated gas will be able to escape from the waste domain without expelling any contaminated pore water. Additionally, sufficient voidage remains after grouting to prevent possible swelling of the waste from exerting excessive pressure on the caisson walls.

There is no specific safety function directly related to mechanical stability of the concrete structures in the long term. However, mechanical stability is relevant to safety in so far as formation of fractures will increase the hydraulic conductivity and may cause groundwater flows to become sufficiently high that the retention of radionuclides is compromised.

3. Caisson Construction Tests

3.1. General

This section provides a brief summary of the main reports that describe tests of the concrete proposed for use in the caissons, and the proposed construction process, providing a review of the main details for each report.

3.2. Review Summary of Document TR-18-12

3.2.1. General

The document TR-18-12: "Concrete Caissons for 2BMA: Large scale test of design and materials" (Mårtensson and Vogt, 2019) was reviewed as part of the work scope. A brief summary of pertinent details is provided herein.

The report describes the general prerequisites and requirements for concrete development for use in construction of the 2BMA caissons:

- The report states that casting of the concrete should be carried out using formwork without the use of formwork tie rods, to avoid the risk of localised channels of higher hydraulic conductivity.
- Casting should be carried out in one continuous sequence to avoid the formation of construction joints.
- Concrete should be designed to minimise shrinkage and cracking during operation and construction.
- The concrete should be pumpable for at least three hours after mixing, it must be workable to ensure ease of compaction, but also be stable to avoid segregation during transport and placement.
- The setting time of the concrete must be formulated to avoid high formwork pressures during casting of the walls, but long enough to allow for adequate placement and compaction, avoiding unintended joints.
- Steel reinforcement is not to be used in the caissons.
- The compressive strength of the concrete should exceed 50MPa at 90 days from casting.
- The tensile strength of the concrete should exceed 2.5MPa.
- Internal strain must be minimised to limit the risk of cracking.
- The concrete should have low porosity to improve durability.
- The cement type of the concrete should be sulphate resistant, low alkali and have low heat development.
- Low cement content is preferred and should be optimised for low heat and good workability.
- Water/cement ratio should not exceed 0.50.
- Modern admixtures may be used but should be limited.
- Solid additives, such as fillers, may be used.

The main purpose of the work presented in the TR-18-12 report was the following:

- A mix design was selected for the large-scale production testing in order to determine that the concrete for caisson construction works could be produced with consistent quality.
- Longer term properties of the concrete were examined and the effect of the climate in the repository assessed.
- Reduction of the restraint between the foundations and the base of the caisson was investigated.
- Investigation of the properties of the joint seal between the slab and walls.
- Based on the above criteria, a range of concrete types were developed and tested. Table 1 presents the concrete developed and selected for the trials. The mix was based on development work described in Lagerblad et al. (2017).

Table 1: Proportions of selected caisson concrete mix

Constituent	Description	Proportions (kg/m^3)
Cement	Degerhamn Anläggningscement/ Cementa AB	320
Filler 1 (2µm)	Omyacarb 2GU/ Omya	130
Filler 1 (10µm)	Myanit 10 / Omya	33.3
Water		156.8
Coarse aggregate (16-22mm)	Crushed rock	393.3
Coarse aggregate (8-16mm)	Crushed rock	425.7
Coarse aggregate (4-8mm)	Crushed rock	92.0
Fine aggregate (0-4mm)	Crushed rock	840.9
Superplasticiser	Master Glenium Sky 558 / BASF	1.30
Superplasticiser	Master Sure 910 / BASF	1.70
Retarder	Master Set RT 401 / BASF	0.96

3.2.2. Preparation of the Test Area

The caisson trials were carried out in a side tunnel (TAS05) at the Äspö Hard Rock Laboratory, which is approximately 3100m from the entrance at a depth of around 420m. The dimensions of TAS05 are approximately 16m in length, 6m in height and 4.5m in width. Plastic sheeting was placed on the foundation slab to reduce any restraint to the base.

The foundation base slab of the test site was formed of reinforced concrete (15m x 4.05m x 0.5m). Conventional formwork was used due to small space constraints, using formwork tie rods.

Joint seals of copper alloy sheet were installed when casting the base and the wall separately. Figure 3 below shows the plan of the walls trialled and the sections showing the two walls that were cast, one with and one without waterstops.

Component 1 was cast first without any waterstop materials and was cast continuously with the base slab (see also Figure 4), with details of the copper waterstops to be embedded at the joint between the base and walls for the second wall (Component 2).

The walls were instrumented with temperature and strain gauges. Formwork pressure cells were also installed, along with displacement transducers to measure dimension change and humidity meters.

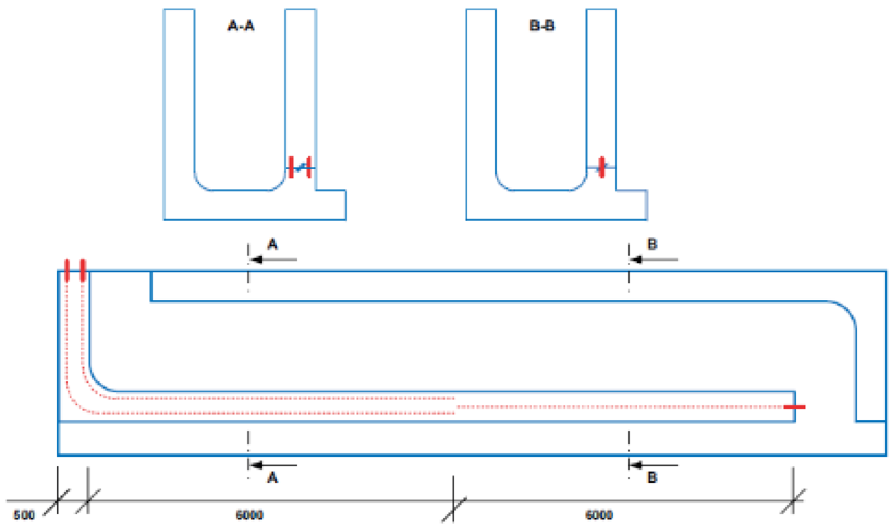

Figure 3: Dimensions of components 1 and 2. Width of base slab 3m. Red lines indicate joint seals. Placement of joint seals in component 2 (Red lines) (SKB TR-18-12)

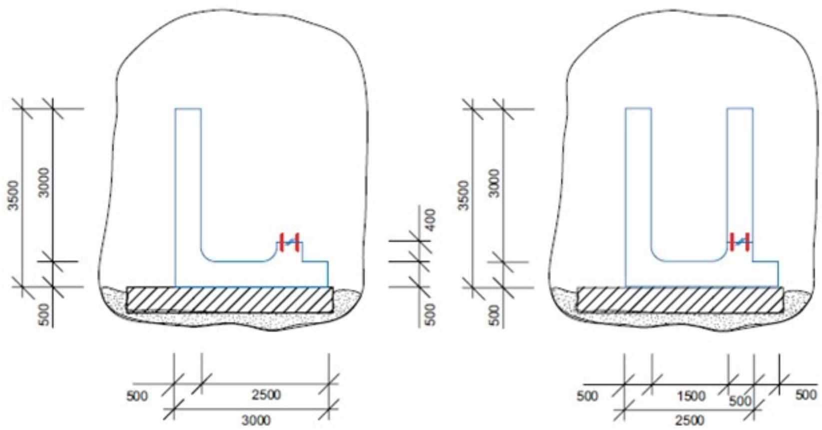

Figure 4: Continuous casting of the L-shaped base-wall. The initial cast using the concrete was the continuous castoff the base and the wall. Second wall placed with copper alloy waterstops (SKB TR-18-12).

Prior to casting, the pre-constructed concrete foundation slab of the TAS05 tunnel (Mårtensson and Vogt, 2019; Section 4) was covered with a reinforced plastic sheeting (termed foil). This was placed in order to reduce the restraint between the foundation and the base slab, thus reducing the risk of cracking.

3.2.3. Issues During Construction of 1st Wall Section (Component 1)

It was reported that the concrete from all seven truckloads for the first base-wall section was very stiff (i.e. low workability). The transport time from the batching plant to the site was around 90 minutes. Concrete properties at the batching plant are shown in Table 2 below.

Table 2: Concrete properties from plant production control.

Truck #	Time	Temperature (°C)	Slump (mm)	Air content (%)
1	17:00	–	210	2.4
2	17:30	14.5	220	2.5
3	18:05	14.8	240	2.4
4	18:30	19.1	230	3.0
5	19:20	17.0	240	2.5
6	20:50	15.9	230	2.6
7	22:10	14.6	240	2.4

Table 3 below shows the properties of the concrete received at the point of placement.

Table 3: Concrete properties on arrival at TAS05.

Truck #	Time	Temperature (°C)	Slump (mm)	Air (%)	Comment
1	18:40	12	210	1,8	At 19:00, slump 210 with about 2 m³ used.
2	19:30	16	100	Not measured	Pump stop. Adjustments according to Table 5-4. Rejected with about 2 m³ left in the truck.
3	20:15	16	90	Not measured	Concrete too stiff. Rejected upon arrival. Cube K1 + K2.
4	21:00	Not measured	120	Not measured	Concrete very stiff upon arrival. Slump value after adjustments according to Table 5-4. Cube K3 + K4.
5	22:30	Not measured	70	Not measured	Adjustments according to Table 5-4. Cube K5+K6.
6	23:30	17	170	2,7	Adjustments according to Table 5-4. Cube K7+K8.
7	00:45	Not measured	150	Not measured	Casting of large cube and specimens for shrinkage measurements Cube K9–K12.

Comparison of the Table 2 and Table 3 shows that a significant loss in workability (consistence) of the concrete was experienced during transit from the batching plant to TAS05. Adjustments were required to re-temper the concrete and reinstate the workability for adequate placement. The addition of both water and admixtures was carried out, leading to water/cement ratios in excess of the 0.50 requirement in some cases. Table 4 presents some details of the adjustments.

Table 4: Adjustments made to improve concrete workability.

Truck #	Water (amount added)	Additive (l/m³ concrete)	Comment
1			No adjustments were made
2	50 litres added: slump 90 mm 20 litres added with 4m³ left in truck	2 litres Master Glenium SKY558 0.5 litre Air entraining agent	Rejected with about 2 m³ left in the truck.
3			Concrete too stiff. Rejected upon arrival.
4	A total of 160 litres added	2 litres Master Glenium SKY558.	Concrete very stiff upon arrival.
5	A total of 120 litres added upon arrival		
6	A total of 140 litres added upon arrival	2 litres Master Glenium SKY558	
7	A total of 120 litres added upon arrival. (slump:150 mm)		

For the continuous cast (i.e. Component 1), the base slab was covered with a lid, to ensure the concrete in the slab was not displaced when casting the wall concrete. As a result, the surface finish of the base slab was poor, showing imperfections due to entrapped air and water during the cast (Figure 5), and casting the slab was reported as very time consuming.

Figure 5: Images showing imperfections on surface of the base slab (SKB TR-18-12).

Figure 6: Section 1 after removal of formwork (SKB TR-18-12).

Issues with the loss of workability (explained as early setting of the concrete) were reportedly due to the use of warm water at the production plant, the use of limestone fillers, and the insufficient dosage of set retarding admixtures. Despite the issues during casting, the walls of the first component exhibited smooth surfaces finishes with minimal imperfections (Figure 6). The base slab was uneven and full filling of the form was not achieved.

Filling of the copper joint seals was generally good, with few imperfections.

3.2.4. Casting of the Second Wall Section (Component 2)

Casting of the second wall section was reported to be considerably easier, due mainly to the large addition of set retarding admixture to the concrete mix (increased from 0.96 L/m^3 in the original mix to 4.16 L/m^3). The spatial relationship between Component 1 and Component 2 is shown in Figure 7 and Figure 8.

Figure 7: Overview of wall Component 1 (left) and wall Component 2 (right) (SKB TR-18-12).

The form filling of the walls was reported to be good for the second wall component. Filling of the joint seals was also completed.

Figure 8: Interior of the trial structure with Component 2 on left and Component 1 on right. Water in base from groundwater leakage from tunnel roof (SKB TR-18-12).

3.2.5. Properties of the Hardened Concrete

The material properties of the continuous casting concrete (Component 1) demonstrated a mean compressive strength of 49 MPa at 90 days, with a mean strength of 67.7MPa at 6 months. The tensile strength of the concrete at 6 months was 2.64 MPa with a standard deviation of 0.55 MPa. Total shrinkage of the concrete was measured as 0.4% after 112 days.

The mean compressive strength for the second cast section at 90 days was 49.6 MPa, and 67.8 MPa at 6 months. The 6-month tensile strength was measured as 2.33 MPa, below the 2.5 MPa requirement. Concrete shrinkage was measured as 0.3% after 112 days.

A total of 10 additional cores were removed from the finished sections and further tests were carried out. Compressive strength was slightly lower than cube samples at 6 months, at 61.5 MPa.

Hydraulic conductivity tests were carried out on core samples, However, the test method (permeability measurement by flow method) used was unable to detect water throughflow.

3.2.6. Monitoring of the Sections

Both wall sections and slab were monitored for a period of around 4 months after casting. The peak temperature observed during hydration of the concrete was 41°C for the first wall section and 38°C for the second wall section, reducing to ambient temperatures after a few weeks.

Compressive strains were observed during hydration in both sections, as would be expected due to increased heat and expansion of the concrete. However, tensile strains were not observed in the sections.

No cracks were observed in any of the sections over the period.

The reporting does not make it clear whether the strain gauges were corrected for temperature, which may explain the results.

3.2.7. Stress Test

The sections were enclosed and force-dried with two large dehumidifiers, to a relative humidity of around 60% for around four months. Surface relative humidity in the concrete reduced to around 80%, where the centre of the concrete was unchanged at around 95%. Average shrinkage was reported as 0.08% in the horizontal plane and 0.04% in the vertical plane. No cracks were observed.

3.2.8. General Conclusions

From the review, the general conclusions and discussion of the report summary is as follows, with the reviewer's comments given in bold.

1. The report summary states that mixing of large amounts of concrete in a concrete production plant was successfully carried out and the properties of the fresh concrete immediately after mixing were according to the requirements. **This is agreed, although significant loss in workability of the concrete caused problems in placement of the mix, particularly for the base.**
2. The report states that the control of setting of concrete containing large amounts of fine-grained limestone filler, such as the concrete used in these experiments, presents a challenging task, due to the fine-grained limestone in the mix that acts as an accelerator. **This is agreed, but this issue is considered a potentially serious concern to the mix design of the concrete in relation to its ease of placement, compaction and finishing, and is discussed further in Section 4.2.**
3. Due to the concrete setting issue, the report states that a concrete production unit in close proximity to the site of the casting needs to be considered in future casting experiments and casting of the caissons in 2BMA. **This is agreed, along with concerted effort in improving the fresh properties of the concrete mix.**
4. The report concludes that, due to the concrete issues, casting the base slab and walls in one cast is very challenging. The results presented in the study recommend that base slab and walls are cast on separate occasions**. This is agreed with the current mix. Even with an improved mix, generally the quality of the construction is at higher risk when casting in one episode.**
5. The report states that the above conclusion is also supported by the fact that the hydraulic properties of the joint were similar to that of the bulk concrete. **This is not agreed, as evaluation of the concrete at the joint was not considered conclusive.**
6. The foundation used in these experiments comprised a thick reinforced concrete slab with a smooth surface finish on which a reinforced plastic foil

had been placed in order to promote unrestrained shrinkage of the concrete structures. From the results obtained up to 1 year after casting of Component 1, this combination has fulfilled these requirements and no cracks have yet been identified. This in spite of the concrete structures having been deliberately dried out at a relative humidity (RH) of 60 % and a temperature (T) of about 18 °C over a period of about 5 months. **This is agreed.**

7. The properties of the hardened concrete were for both sections within the requirements, presenting a high strength and low hydraulic conductivity in combination with low shrinkage and low levels of strain. **This is partially agreed, as hydraulic conductivity was not adequately established.**
8. The hydraulic conductivity of joints and the interface between the joint seal and the concrete was found to be closely similar to that of the bulk concrete. This indicates that casting the base slab and the walls separately can be a suitable way forward in order to avoid a complicated casting process with a suspended formwork. **It is not agreed that the hydraulic conductivity of the joints was similar to the bulk concrete. Casting the slab and walls separately is considered a viable option with additional consideration required for waterproofing design. Further discussion on hydraulic conductivity and joint review is given in Sections 4.2 and 4.4.**
9. The report states that the results presented indicate that the previously developed concrete can be used to cast the caissons for disposal of radioactive waste in 2BMA. However, further development of a method for concrete mixing and a method for casting needs to be considered. **This is not agreed. Potentially serious concerns in relation to the concrete mix design put <u>consistent quality</u> of caisson construction at risk.**

3.3. Review Summary of Document TR-20-09

3.3.1. General

The document TR-20-09: "Concrete caissons for 2BMA. Large scale test of design, material and construction method" (Mårtensson and Vogt, 2020) was reviewed and pertinent information is summarised in this section.

This report describes the casting trials of a concrete caisson at the TAS08 site area at the Äspö Hard Rock Laboratory, including the experiences of the use of a mobile concrete batching plant located close to the entrance of the Äspö tunnel.

The dimensions of the caissons reported here are different to those described in TR-18-12. Instead of the dimensions of the caissons reference design of 16.2m x 16.2m x 8.4m, with a thickness of the walls and base slab of 500mm in TR-18-12, this report describes the reference design for caisson dimensions as 18.1m x 18.1m x 9m, with the thickness of the walls and slab at 680mm and 600mm, respectively. It is therefore assumed that this is the updated reference design.

The main purpose of the TR-20-09 report is described as follows:

- To conduct a large-scale concrete production test to verify consistent quality.
- To perform a production scale test casting of a concrete caisson representative of the concrete caissons for 2BMA.
- To verify that restraint between the foundation and the caisson base slab can be mitigated by founding the caissons on a smooth concrete slab covered with plastic sheet in order to reduce the adhesion between these parts.
- To investigate the effects of warming the base slab prior to casting of the walls as a means to mitigate the effect of temperature shrinkage of the walls after casting.
- To study the hydraulic properties of the joint between the base slab and the walls of the concrete caisson.
- To perform long term follow-up of the properties of the caisson and the effect of the climate in the repository on these properties.
- To develop methods for sealing the holes from the tie rods and study the hydraulic properties of the sealed holes.

3.3.2. Caisson Design Details

Due to space constraints within the TAS08 cavern, the dimensions of the trial caisson were designed to represent a similar length-height ratio of an actual 2BMA caisson design. Thus, the length of the caisson was 16.74m, the width 9.36m and the height of the walls 4m. The thickness of the walls was 0.68m, and the base slab had a thickness of 0.6m.

The use of unreinforced concrete stipulated in the prerequisites of the design brief re-quires the need to reduce peak stresses. Therefore, the basic design stipulates the casting of the joint between the base and the walls is partly inclined, and the inner corner of the wall would be chamfered 150mm at an angle of 45 degrees.

Joint seals were also employed, utilising 300mm wide copper sheet, as in the previous trials. One joint sheet was installed, as shown in Figure 9 below.

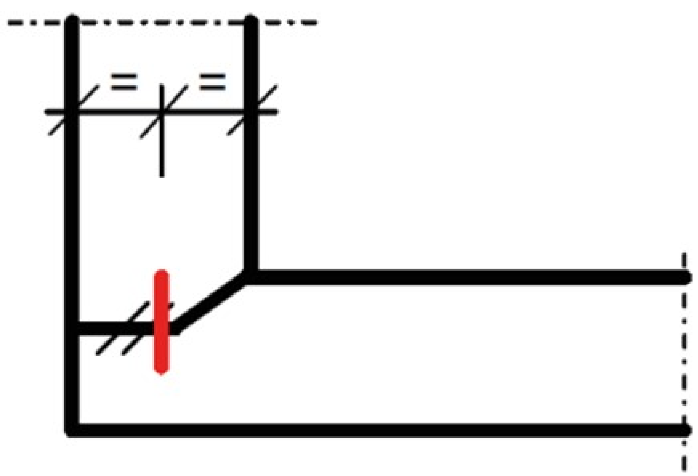

Figure 9: Placement of joint seals at the slab/wall joint (SKB TR-20-09).

Conventional formwork was used, with the use of formwork tie rods that were placed inside plastic tubes.

The trial works were instrumented with temperature and strain gauges. Formwork pressure cells were also installed, along with displacement transducers to measure dimension change and humidity meters (see Section 3 of Mårtensson and Vogt (2020) for details).

3.3.3. Preparation of the Test Area

Preparations within TAS08 comprised the casting of a foundation slab on which the caisson could be erected. After rock debris removal and infill works to fill large holes, the foundation was cast. However, the concrete (a conventional C40/50 concrete) stiffened too quickly and finishing operations were not possible. The foundation slab was subsequently ground mechanically to the required flatness and finish (Figure 10).

Figure 10: Final surface finish once the test area was complete (SKB TR-20-09).

3.3.4. Concrete Production Plant

The selected concrete mix as described above and presented in Table 1 was employed for these trials.

A mobile batching plant was commissioned adjacent to the tunnel entrance. Section 5.2 of Mårtensson and Vogt (2020) describes the details of the mobile plant, and the powder, water and aggregate storage areas. Mobile mixer trucks were used to mix the concrete (Figure 11), before being transferred into a larger concrete mix truck for transportation.

Figure 11: Cement and limestone powder silos (SKB TR-20-09).

Batch handling of the materials varied. Aggregates were batched using the truck mixer's own loading bucket. Cement and 2GU limestone powder were fed from the silos via screw feeders. The Myanit 10 powder was manually added to the batching bucket of the mixer truck.

Figure 12 below illustrates the batching operations using the self-loading trucks. Once all components had been fed into the mixing truck, the concrete was mixed for around five minutes before being poured into the transport truck (Figure 13). The slump and the temperature of the concrete were measured.

Figure 5-10. Feeding aggregates (0–4 mm) into the truck mixer.

Figure 12: Feeding aggregates and powders into the truck mixer (SKB TR-20-09)

Figure 13: Truck mixer positioned for pouring concrete into the transport truck (SKB TR-20-09).

3.3.5. Concrete mix adjustments

Adjustments to the concrete mix were required during commissioning of the batching plant. The first four loads of batch concrete were consistently too stiff, progressive increases in water and some admixtures were needed to achieve the required fresh properties.

The final mix ended up containing more water than anticipated in the design (up to 0.53 instead of 0.50). The report also detailed a precise addition of constituents required for the mixing.

It was concluded in the reporting that the mixers used were not considered to be well adapted to the type of concrete mixer used.

The mixing process appeared to take up to 2 hours to fill the transport truck from two mixing trucks. This is considered by the reviewer to be an excessive timeframe, which increases the potential for problems with the stiffening fresh concrete.

3.3.6. Beam Casting Details

In order to check the function of the strain gauges, insulated and uninsulated beam moulds were cast, using both the 2BMA mix and a standard concrete mix. Internal strains were monitored for all four configurations.

The measured strains were reported to be far higher than expected due to the unrestrained situation in the moulds, which could not be explained.

Although reported that compressive strains in the beams were unexpected, in fact they are likely to be a consequence of early thermal behaviour in a restrained condition. It is also important to note that the stiffness (modulus) and associated creep capacity of the concrete is changing mechanical properties, causing residual strains in the element. It would be possible to calculate the restraint by plotting the strain against the temperature.

3.3.7. Base Slab Casting

The base slab was cast on plastic sheeting to eliminate any restraint from the foundation slab. The base slab form work is shown in Figure 14. During casting, a total of around 100 m^3 of concrete was produced and transported to the base slab location. The base slab took around 8 hours to complete, with an average time for batching of concrete to placement of 7 m^3 loads at around 90 minutes.

Concrete slump was consistent at between 200 mm and 240 mm. The first load was reported as hard to pump, and for the 10th load, around 4 m^3 of concrete was discarded due to pumping problems. The finished slab was power-trowelled and covered with plastic sheeting.

Figure 14: Base slab formwork ready to receive concrete (SKB TR-20-09).

The base slab surface finish appeared to show a coarse finish (Figure 15), which although not reported to be an issue, is considered a result of the fast drying of the concrete mix.

Figure 15: Coarse surface finish of the base slab, indicating tearing of the drying surface during trowelling (SKB TR-20-09).

Average compressive strength results were 47.1 MPa at 28 days, and an average of 64.8 MPa at 6 months. These are slightly lower than the previous trials, possibly due to the slightly higher water cement ratio of the concrete.

Splitting tensile strengths (as opposed to direct tension testing in the previous trials) showed an average of 4.7 MPa. Drying shrinkage tests showed total shrinkage of 0.38% at 224 days.

No cracks were observed within the base slab.

3.3.8. Wall Casting

Casting of the walls was prepared by erecting conventional formwork. Heating mats and insulation were also installed on the base slab prior to casting. The reason for this was to reduce the restraint of the base slab to the walls by heating it as the wall concrete is poured and then allowing the slab to cool at the same rate as the walls. The similarly contracting walls and base reduces the restraint at the joint, thus allowing similar thermal behaviour of the slab and the walls and reducing the risk of cracking.

Around 150 m^3 of concrete was produced for casting the walls, which took around 10 hours to complete the wall cast. The concrete mix was adjusted slightly, by removing the set retarding admixture as the ambient temperatures were low (around 3°C) meaning the concrete placement temperature was reduced and therefore setting times were extended. Concrete placement rate was reported to be around 14 m^3 per hour.

Formwork pressures were measured, up to 28 kPa at around 0.48m from the base of the formwork, which was around half of the formwork design pressure.

The concrete caisson after removal of the formwork is illustrated in Figure 16.

Figure 16: The caisson after formwork removal (SKB TR-20-09)

There was some reported evidence from detailed inspection of a slight colour change in the concrete, due possibly to the differences in batching between the two batching truck mixers. This may demonstrate the differences in the batching tolerances between the mixers, and perhaps the less efficient mixing compared with forced action mixing.

Inspection of the walls following removal of the formwork showed generally good compaction, other than isolated areas where poor intermixing of layers has caused some honeycombing and pour lines (Figure 17). This is likely to be caused by lack of compaction from the internal pokers.

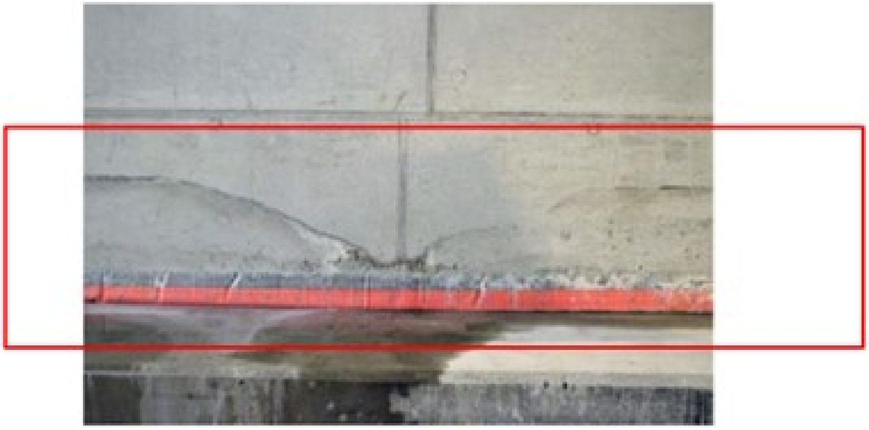

Figure 17: Poor compaction at the slab / wall joint (SKB TR-20-09).

Compressive strengths at 28 days were measured at an average of 45 MPa. A comparison between the base slab and the previous trials indicates higher variation using the mobile plant than the established batching plant.

Compressive strengths at 6 months are around 56 MPa on average, with a large range between maximum and minimum values (49 to 61 MPa). Tensile splitting strength of retrieved cores showed an average of 3 MPa at 6 months which included samples from the joint. For the bulk concrete, the average was 3.8 MPa. This is markedly lower than the base slab average, at 4.7 MPa.

Specimens used for shrinkage testing showed total shrinkage of around 0.36% at 224 days.

Cores were also taken to test for hydraulic conductivity of the bulk concrete at the joint. Four cores were taken, two of which were taken at the joint between the wall and the base slab. Measured hydraulic conductivities are given in Table 5.

Table 5: Measured hydraulic conductivities of concrete core samples.

Specimen	Hydr. cond. [m/s]	Direction of drilling	Comment
F08000008a	5.4×10^{-8}	Across the core	In the joint
F08000008b	3×10^{-11}	Across the core	In the joint
F08000011 IN	1×10^{-11}	Along the core	Centre of wall
F08000013	3×10^{-12}	Along the core	Centre of wall

Based on the results in Table 5, the report states that the hydraulic conductivity of the joint is similar to that of the monolithic concrete and that consequently joint seals would not be required. However, the report also states that the most likely reason for the higher result at the joint (8a) is likely to be due to poor surface preparation of the joint prior to casting (which could potentially occur during construction). Additionally, the average results between the two test types show major differences between the two sets of results (see Section 4.4.2).

3.3.9. Tie Hole Grouting

As noted in the project requirements, formwork should be constructed without the use of tie rods. This is, however, difficult to achieve in this situation and so trials were conducted on filling the tie rod holes. Further discussion on this is provided in Section 4.5.

Various trials were carried out to grout the tie holes once the metal tie rods and plastic inserts were removed (Figure 18). It was concluded that the plastic inserts were difficult to remove, and grouting of the holes was partially successful, with only a few holes fully filled and with permeability to that of the surrounding concrete (Figure 19).

Additional trials were found not to be compatible with the design, and further development of the concept is required.

Figure 18: Grouting hole with hand-held injection pump (SKB TR-20-09).

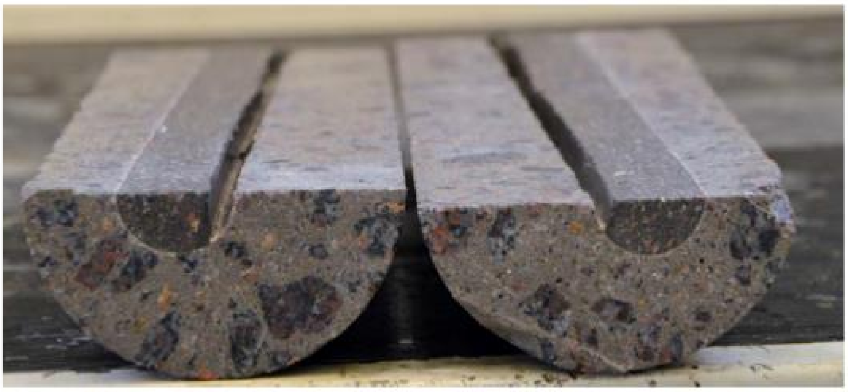

Figure 19: Cores through tie rod holes show incomplete grouting (SKB TR-20-09).

3.3.10. Base Slab Monitoring

With a placement temperature of around 16°C, the concrete in the base slab reached a peak temperature of around 41°C at about 20 hours.

Internal strains showed an increase in compressive strains during hydration, which reduced over time.

Heating of the base slab prior to casting the walls to a maximum of 43°C induced a change in dimension of around 2.4 mm.

No cracking was observed in the slab.

3.3.11. Wall Monitoring

The temperature of the walls reached a peak of around 43°C at around 35 hours. The temperature of the walls returned to ambient after around 25 days.

Internal strains in the walls showed an increase in compressive strains during hydration, which reduced over time. No cracking was observed in the walls.

3.3.12. General Conclusions

From the review, the general conclusions and discussion of the report summary is as follows.

1. The work presented in the report detailed the construction and long-term monitoring of the properties of the concrete trial caisson. The dimensions correspond to ¼ of the caissons in the future 2BMA, with concrete thicknesses the same as planned for the 2BMA caissons.
2. Preparation works for the foundation slab were completed, after the surface needed to be ground smooth due to poor surface finish. Low friction plastic sheeting was placed between the foundation slab and the base slab to reduce the risk of base slab cracking. **The use of plastic sheeting is considered good practice for this purpose**.
3. Caisson casting was divided into two steps, where the first comprised casting of the base slab and the second step the walls.
4. The concrete used in these experiments had previously been developed and tested by Lagerblad et al. (2017) and is the same as used in the previous trials. The large amounts of limestone fines are reported to be required to obtain the desired workability. **The statement that the desired workability may have been achieved is agreed, although the reviewer contests that workability retention is a potentially serious concern to construction risk due to inconsistent mix quality at the point of placement**.
5. The report states that the concrete for the base slab was good, despite also reporting that the concrete consistently required additional water to be added in order for it to flow. **The reviewer notes that occasional rejections of concrete were required, typically due to construction delays and poor workability retention of the concrete.**

6. Casting of the walls was reported to be completed successfully. The base slab was pre-heated to reduce the restraint of the wall concrete from the base, and therefore reduce crack risk. **This is generally considered useful practice, albeit labour intensive. Further design considerations may mitigate the need for this.**
7. Localised low compaction of concrete was reported in the walls, which was probably due to workmanship practices. **This is agreed, although issues with workability retention may have made compaction more problematic.**
8. Batching of the concrete using mobile plant is considered rather slow, and it is concluded in the report that the batching facility employed was not matched for its intended purpose. **This is agreed.**
9. Mechanical properties of the hardened concrete were shown to be acceptable in relation to the requirements.
10. Long term monitoring of the concrete did not provide the information anticipated, although no negative aspects in structure behaviour were reported. **This is noted.**
11. No cracks were observed in the completed structures.
12. Sealing of the tie rod holes was trialled. It was concluded that no clear or reliable method has been found to effectively fill the tie holes with material of the same hydraulic conductivity as the bulk concrete. **The reviewer considers that SKB needs to give further consideration to tie rod design and filling trials.**

4. Discussion of Caisson Construction

4.1. General

This section examines various aspects of the trials reports, which are considered the most pertinent issues for further appreciation and discussion in the development of optimised solutions for the 2BMA caisson construction.

The principal topics are:

- Concrete mix design;
- Cracking risk and restraint;
- Joint design review and risk;
- Tie holes; and
- Embedded reinforcement.

4.2. Concrete Mix Design

4.2.1. General

The main reports reviewed (Mårtensson and Vogt, 2019, 2020) conclude that the selected concrete mix design (Table 1 and Table 6) can be used on a production scale to cast large concrete structures. However, it is questioned herein whether the concrete mix is in fact optimised for the purpose of caisson construction. Concerns arise from the implementation of the selected concrete mix based on the reported observations during the trials (see Section 3.2.8 and 3.3.12).

Both reports mention that the mix does not retain its fresh properties readily (that is, the mix's ability to retain its workability). As a result, a number of batches were rejected and significant adjustments to the batched concrete were required to ensure the concrete maintained its workability (i.e. consistence) during transport and placement. The principal reason for this is reported to be the excessive use of limestone powder within the mix. Consequent problems with water demand, cohesion and retention of workability follow from this, as well as a certain degree of impracticalities in batching the mix.

This is reported to be due to the fact that the fine-grained limestone powder in the mix acts as a set accelerator in the mix.

Further review and discussion are provided below.

4.2.2. Review of Mix Design

Table 6 below represents the selected concrete mix (from Table 1 above), with the addition of a new column to describe the constituent ID.

The main reason for the inclusion of limestone powder filler in the SKB trials is understood to be that it provides an optimised grading curve that minimises the water demand of the mix, hence reducing the overall shrinkage of the concrete mix.

Substantial work on the rheological properties of the concrete mortar fractions is reported in Lagerblad et al. (2017). It was concluded that limestone filler concretes provided suitable rheological properties, whilst minimising the effect of autogenous (i.e. chemical) shrinkage when compared with additions of reactive silica. Lower water contents dictate ultimate shrinkage rates.

Small-scale trials of the selected mixes described in Mårtensson and Vogt (2019) reported that the limestone filler was acting as a set accelerator to the mix, and retarding admixtures were recommended for further trials.

During the caisson construction trials, it was observed that significant issues were encountered with the fresh properties of the selected concrete mix. Nevertheless, use of the mix continued to the second set of trials. Both reports (Mårtensson and Vogt, 2019, 2020) acknowledge the problems with retaining workability of the concrete, which was mitigated only by addition of water and set retarders. Again, this appears to be due to the use of the cement binder and powder combinations.

Table 6: Proportions of the selected caisson concrete mix.

Constituent ID	Constituent	Description	Proportions (kg/m^3)
1	Cement	Degerhamn Anläggningscement/ Cementa AB	320
2	Filler 1 (2µm)	Omyacarb 2GU/ Omya	130
3	Filler 1 (10µm)	Myanit 10 / Omya	33.3
4	Water		156.8
5	Coarse aggregate (16-22mm)	Crushed rock	393.3
6	Coarse aggregate (8-16mm)	Crushed rock	425.7
7	Coarse aggregate (4-8mm)	Crushed rock	92.0
8	Fine aggregate (0-4mm)	Crushed rock	840.9
9	Superplasticiser	Master Glenium Sky 558 / BASF	1.30
10	Superplasticiser	Master Sure 910 / BASF	1.70
11	Retarder	Master Set RT 401 / BASF	0.96

Following the trials there appears to have been little consideration given to addressing the suitability of the mix design itself. Much focus has been applied to the mitigation of the shrinkage of the caisson concrete. Although this is an important requirement of the design, too much focus on one aspect of the mix may impair other requirements. In this case, it is considered that constructability may have been overly compromised at the expense of shrinkage design.

Limestone fines are produced by milling and classification resulting in powders with a high specific surface area.

The use of ground limestone in cements has been allowable in European standards since the production of EN 197-1 (British Standards Institution, 2000). Two designations are stated in the Standard, CEM II/A-L (or LL) (6-20% limestone) and CEM II/B-L (or LL) (21-35% limestone). This material is interground with the cement or added as a pro-portion of the cementitious binder. The requirements for the limestone are:

- $CaCO_3$ content ≥ 75 per cent;
- Clay content, as determined by the methylene blue test shall not exceed 1.20 g/ 100 g; and
- Total Organic Carbon (TOC) shall not exceed 0.20 per cent for LL limestone or 0.50 per cent for L limestone.

Alternatively, the use of limestone as a filler material for use with Portland cement is also allowable in the UK, in accordance with BS 7979 (British Standards Institution, 2016), to produce the fines materials themselves, rather than the interblended materials with cement. The compositional requirements for limestone fines are like those in EN 197-1 (British Standards Institution, 2000).

Specific requirements for certain physical properties are included. Limitations on compressive strength, initial setting, soundness and fineness of the material are given in the standards, based on a notional proportion of limestone fines of 20%.

Limestone filler was initially considered to be an inert filler; however, it is now accepted as contributing to the hydration process by the formation of calcium monocarboalumniates (C3A. $CaCO_3$. $11H_2O$) (Newman and Choo, 2003). (Note. C3A is generally accepted shorthand for tricalcium aluminate).

Whilst it is acknowledged that limestone powder provides certain advantages to concrete, on review of the selected mix, it is considered that the concrete contains excessive proportions, despite showing promise in laboratory development conditions. The proportion of limestone powder in the current selected concrete mix is around 34%. This represents the upper envelope of the CEM II/B-L designation, the majority of which is a very finely divided size-fraction (i.e. 2 micron maximum size), thus producing a system with very high specific surface.

Typically, addition of limestone powder in proportions of around 15-20% replacement of Portland cement will provide no disbenefit to the mix. Moreover, the inert filler will act as a latent hydraulic material at these levels and provides nucleation sites within the paste fraction for further hydration.

Beyond this typical level of replacement, limestone powder is regarded as a filler and may have an adverse effect on concrete properties with increasing levels of replacement. In addition, its finely divided nature may increase the water demand of the concrete in order for the paste to flow around the aggregate, thus potentially increasing ultimate shrinkage of the concrete (due to higher water demand).

Matthews (1994) found that a 0.01 increase in water/cement ratio was needed for 5% limestone replacement, and a further 0.01 increase at 25% replacement to achieve the same workability.

Other workers have found improvements in workability with modest additions of limestone fines (up to 20%). Nehdi et al. (1998) found that limestone fine particles reduced the torque viscosity of their concrete to give better early age rheological properties. Advantages in ternary blended systems were also discussed.

4.2.3. An Alternative Approach

It is the reviewer's opinion that the selected mix contains excessive fine limestone material. SKB has not provided adequate justification for the use of this mix and appears not to have considered alternatives that in the view of the reviewer could provide enhanced performance in relation to workability control, crack control and durability. To illustrate this point, potential alternatives are discussed here.

The author has extensive experience in the design and use of concretes using binary and ternary cementitious combinations of binders. It is not clear whether SKB has considered such combinations. A wider review of these alternatives would be beneficial for determining how a fresh concrete mix may be used reliably to construct the 2BMA caissons in certain challenging conditions. Further possibilities are therefore presented below, along with short commentary on comparisons between concretes containing limestone fines and ground granulated blast furnace slag (GGBS).

For information, the principal requirements of the 2BMA vault concrete are set out in Section 3.2.1 of this report.

Concrete mixes using ternary blends of cement have been used in major civil engineering projects in the UK and overseas. These may range from blends of Portland Cement (PC), GGBS, flyash (FA), Silica Fume (SF) and Limestone powder (LP).

Very high-performance mixes (i.e. high strength and high durability) commonly use combinations of PC/GGBS/SF or PC/FA/SF. However, high quality mixes can also be formed from PC/GGBS/LP ternary blends. It is the latter here that the reviewer considers to be a promising alternative to the current high PC/LP mix reviewed.

GGBS is the by-product during the manufacture of iron, which results from the fusion of a limestone flux with ash from coke and the siliceous and aluminous residue remaining after the reduction and separation of the iron from the ore. The slag is rapidly cooled to form a glassy state that is then dried and ground to cement fineness.

GGBS forms a latent hydraulic material that requires activation by either alkalis or sulfates, producing a slowly reacting hydration product that refines the pore structure of the paste to improve long-term hardened properties of the concrete. Its slower rate of reaction means that there is a natural retardation of the mix to hydration, therefore increasing the workability retention and setting time of the concrete. Slower reaction rates also reduce the rate and peak of hydration temperatures in the concrete, thus reducing the risk of early thermal deformations and subsequent cracking risk.

Once hardened, and with adequate presence of moisture, GGBS continues to hydrate and paste structure properties improve. GGBS is known to be particularly resistant to ingress of chlorides and attack from aggressive ground, when compared with similar Portland cement concretes.

The use of small additions of limestone powder to the system would enhance the strength development of the concrete in the short term and benefit the rheology of the concrete, typically providing greater cohesion to the mix without adversely affecting fresh or hardened properties.

Alternative design mix

A typical ternary blend mix design that could potentially meet the design requirements for the caisson concrete is given in Table 7.

Table 7: Potential alternative initial mix design containing PC/GGBS/LP blend.

Constituent	Description	Proportions (kg/m^3)
Portland cement		175
GGBS		215
Limestone Powder		50
Water		175
Coarse aggregate (10/20mm)	Assuming crushed limestone aggregate (Gc85/20) to EN 12620	670
Coarse aggregate (10/4mm)	Assuming crushed limestone aggregate (Gc85/20) to EN 12620	335
Fine aggregate (0-4mm)	Assuming Natural sand (Gf85) to EN 12620	790
Water/cement ratio		0.45
Superplasticiser	Master Glenium Sky 1966 / BASF	As required
Consistence retaining admixture	Master Sure 1970 / BASF	As required
Retarder	Master Set RT 401 / BASF	As required

General Intentions/Assumptions

The mix design above assumes a low Portland cement proportion, with high replacement of GGBS. This mix hydrates more slowly, and thus heat of hydration is reduced, as is therefore early age crack risk. Compressive strengths develop slowly, but continue to increase over the long-term, from a developed pore microstructure.

GGBS concretes are particularly beneficial in concretes exposed to the marine environment. The main reasons for this are their ability for the pore microstructure to improve with time, and the increased binding capacity of chlorides into the paste fraction. Other assumptions include:

- Portland cement minimum grade 42.5N to EN 197-1.
- GGBS in accordance with EN 15167-1.
- Limestone powder filler in accordance with BS 7979
- Limestone powder filler forms part of the aggregate (fines content) rather than part of the binder content.
- Aggregates in accordance with EN12620.
- Admixtures in accordance with EN 934.
- Water in accordance with EN 1008.
- Combined aggregate gradings are proportioned to produce a smooth grading curve suitable for pumping.
- Admixtures are assumed to provide required consistence and retention, although suitable alternatives should be considered as appropriate.

4.2.4. Concrete Properties

The following properties are based on recent test data for an almost identical concrete mix (project undisclosed). Similar results would be expected from initial trials of the mix given in Table 7. The intent here is not to recommend a mix, but to illustrate the concept.

Slump

Initial slump for the suggested ternary mix was targeted at 150mm (S4 slump class). Given the flexibility of the mix, higher slumps would be feasible, and slump retention could be easily adjusted using suitable admixtures.

Bleed

Although GGBS has a similar fineness to Portland cement (designated CEM I in Europe) it increases the setting time of concrete, particularly at high GGBS proportions, and as a result there is an extended time over which bleeding can occur and the total bleed capacity may be increased (Wainwright and Rey, 2000). The UK's Building Research Establishment (Building Research Establishment, 1993) found that increasing the fineness of the limestone fines decreased the tendency for bleeding.

Bleed to EN 480-4 in the above ternary blend concrete blend was measured as 0.018 ml/minute (i.e. 1.6 g maximum at 90 minutes). This would be in line with work by Hooton et al. (2007) that shows the affinity of water absorption on limestone particle surfaces increases with fineness (Figure 20). Here, bleeding rate would correspond to a specific surface area in line with concrete containing 16-25 % limestone fines (approx. 400 m2/kg).

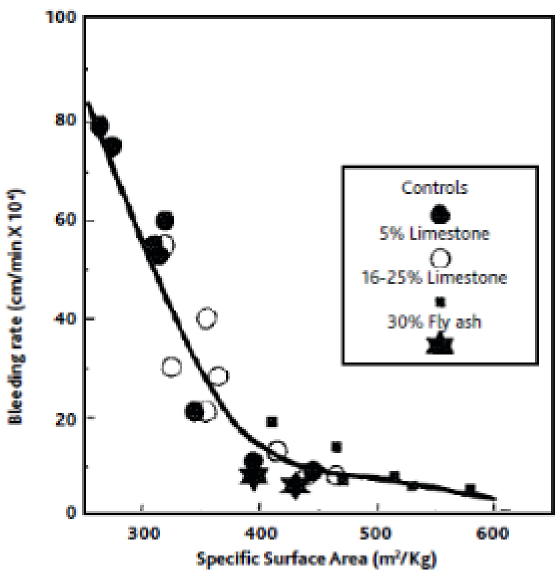

Figure 20: Bleeding rate plotted against specific surface area, which increases with decreasing grain size (Hooton et al., 2007).

The fineness of limestone is a factor influencing set time of cement pastes. Fineness gives much improved particle packing properties which, combined with the fact that it acts as a nucleation site (Soroka and Stern, 1976; Pera et al., 1999) speeds up the hydration process. The magnitude of this effect varies depending on replacement levels and cements used, but in general initial and final set times decrease.

Under typical conditions, the initial set is likely to be extended by about 30 minutes for concrete with 50% GGBS but longer extensions may occur at higher levels and in the winter. The increase in setting time is affected by the initial temperature of the concrete, the proportion of GGBS, w/c ratio and the characteristics of the CEM I, but the greatest influence is the percentage of GGBS, where high replacement levels can markedly increase setting times.

Initial setting times at standard temperature would be expected to be around 3 hours for the suggested mix (195 minutes measured) without the use of consistence retention admixtures or set retarder. Setting times for the suggested mix are typically 30% longer than for a similar Portland cement (CEM I) concrete.

Compressive strength
As aforementioned, the addition of limestone fines improves the particle packing and provides nucleation sites for calcium hydroxide crystals at early hydration ages, accelerating the hydration of clinker particles (Bonavetti et al., 1999), which may result in increased early strength at low levels of replacement (up to 5%). However, as limestone fines are not pozzolanic, they do not produce calcium silicates. Due to the reduction of potential cementing material, strength at later ages is reduced (Sersale, 1992).

Hooton et al (2007) concluded that the trend in tensile and flexural strength is the same as that observed for compressive strength.

The concrete mix suggested above achieved a compressive strength of around 47.9 MPa at 28 days and 50.8 MPa at 56 days, and a flexural strength was measured as 5.9 MPa at 28 days.

This compares favourably with the 2BMA concrete, with a compressive strength of around 49 MPa at 90 days. Slower strength gain of the suggested mix would be feasible by introducing higher replacement levels of GGBS (also thus reducing hydration heat).

Adiabatic heat rise
Theoretically, as they are inert, limestone fines generate no heat in concrete. However, limestone fines do act as nucleation sites for hydration and help increase the heat of hydration when introduced to the CEM I content (Hooton et al., 2009). Vuk et al. (2001) found clinker chemistry had a significant effect on the heat of hydration of cements containing limestone. The higher percentage of the C3S fraction in the cement chemistry, the greater was the increase in energy output noted. Despite this, CIRIA Report C766 (Bamforth, 2018) suggests that limestone fines should be considered as inert with regard to heat generation.

Replacing CEM I with GGBS reduces the temperature rise and can help to avoid early-age thermal cracking. There are a number of factors that determine the rate of heat development and the maximum temperature rise. These include the percentage of GGBS, the total cementitious content, the dimensions of the structure, the type of formwork and ambient weather conditions. The greater the percentage of GGBS, the lower will be the rate at which heat is developed and the smaller the maximum temperature rise. As well as depressing the peak temperature, the time taken to reach the peak will be ex-tended. With thinner sections, significant savings in crack control reinforcement can be achieved even with lower levels of GGBS of 50% or less (Bamforth, 2018).

Adiabatic heat rise in the trial mix was measured using heavily insulated 1m x 1m x 1m concrete cube samples.

The concrete placement temperature measured for the mix was 20°C, and the mix achieved a peak temperature of 53.9°C at around 46 hours. This corresponds to a heat rise of 33.5°C. Further discussion is provided regarding this and early age thermal crack risk in Section 4.3.

Drying shrinkage
A review by Hooton et al. (2009) deduced that when corrected to constant paste content, GGBS concrete shrinks about 3% less than CEM I concrete. This was independent of the GGBS content and water/cement ratio over the ranges typically used in concrete. Such a small difference was not considered significant, and it was concluded that the drying shrinkage of GGBS concrete is approximately the same as for CEM I concrete.

Alunno-Rossetti and Curcio (1997) found that the rate of shrinkage and total amount of drying shrinkage at one year was essentially the same for limestone fines concrete as for CEM I. By contrast, Dhir et al. (2007) reported reduced shrinkage and similar creep in concrete with cements containing 45% ground limestone.

Limestone fines may well therefore produce concretes with slightly lower shrinkage, but GGBS concretes may perform similar to CEM I concretes.

In testing the above suggested mix, combined autogenous and drying shrinkage was measured at 345 microstrain after 56 days. This has been compared with a logarithmic extrapolation of the test data from the 2BMA caisson trials. Figure 21 and Figure 22 illustrate comparisons between the two sets of 2BMA caisson trials with typical test results with similar concretes containing GGBS.

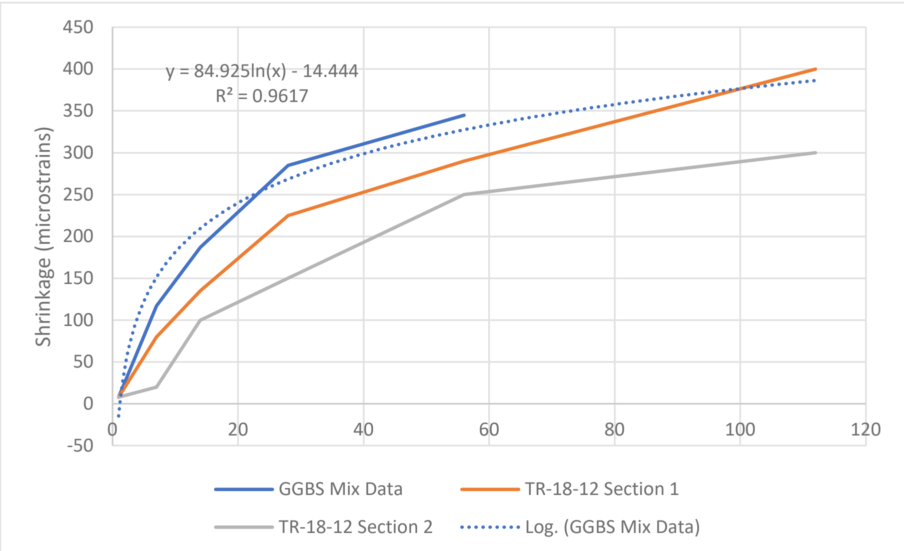

Figure 21: Comparison of total drying shrinkage between caisson trials (x), sections 1 and 2, and total shrinkage of suggested ternary blend mix.

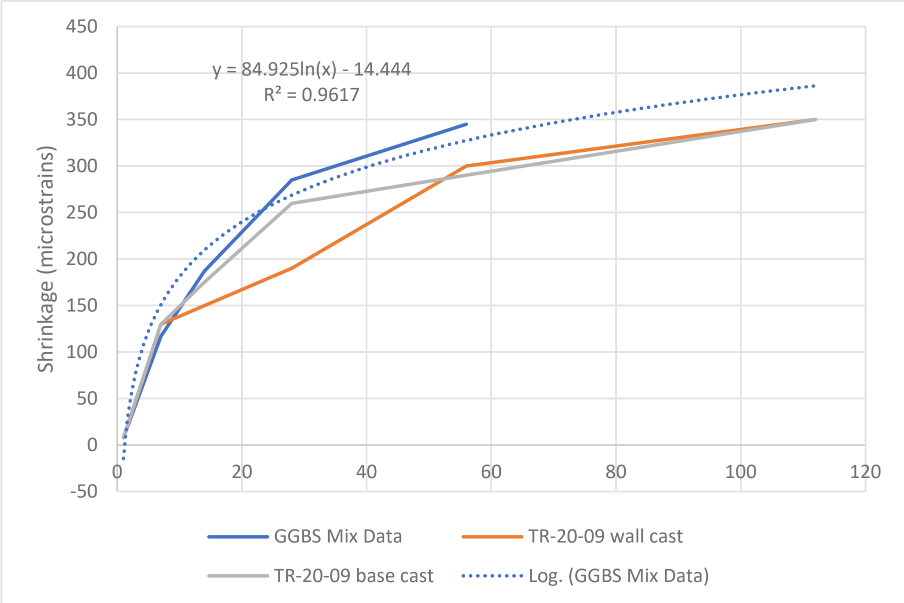

Figure 22: Comparison of total drying shrinkage between caisson trials (2), base slab and wall casts, and total shrinkage of suggested ternary blend mix.

The above results indicate that, on average, the 2BMA mix has slightly lower total shrinkage than the suggested GGBS ternary mix, however, the differences are typically small, around 50 microstrains (i.e. 0.05%). It could be concluded that the differences between the concretes are negligible.

Durability

Due to its finer pore structure, GGBS concrete is substantially more resistant to chloride diffusion than CEM I concrete (Kumar et al., 1987; Page et al., 1981; Decter et al., 1989). Generally, the higher the proportion of GGBS, the greater will be the resistance to chloride penetration. Detailed recommendations for the use of GGBS in environments subject to de-icing salts or sea water can be found in Concrete Society Technical Report 61 (The Concrete Society, 2004).

Hooton et al. (2007) concluded that for concretes produced at the same w/c ratio, incorporation of limestone fines will increase chloride ion penetration but when proportioned to achieve the same 28-day strength, similar performance would be expected. Bentz et al. (2009) used an analytical model of diffusion to show that, for concretes with a w/c ratio greater than 0.4, the addition of limestone fines led to significantly increased diffusion coefficients but this could be offset by a slight reduction in w/c. Thomas et al. (2010) carried out field trails with 12% limestone fines, casting a number of ground-supported slabs. Rapid chloride permeability tests showed no consistent difference in performance between the concretes produced with limestone fines and those with CEM I.

Water permeability

In well-cured laboratory specimens, the water permeability of concretes containing GGBS or fly ash will ultimately be lower than CEM I concrete of the same strength class. This is due to the continued hydration beyond 28 days. Limestone fines have little effect on permeability. Ramezanianpour et al. (2009) found that sorptivity increased with increasing percentages of limestone. Porosity was unaffected with up to 15% limestone in the concrete but started to increase very slightly beyond that figure.

Water permeability was measured in the suggested concrete mix as 1.59 x 10-12 m/s. This compares with the water permeability coefficients reported in Mårtensson and Vogt (2020) for the 2BMA caisson trials as a mean value (within the centre of the caisson wall) (Table 8).

Table 8: Hydraulic conductivity (water permeability) coefficients for the 2BMA trials. Other factors relating to durability

Specimen	Hydr. cond. [m/s]	Direction of drilling	Comment
F08000008a	5.4×10^{-8}	Across the core	In the joint
F08000008b	3×10^{-11}	Across the core	In the joint
F08000011 IN	1×10^{-11}	Along the core	Centre of wall
F08000013	3×10^{-12}	Along the core	Centre of wall

Other factors relating to durability
In addition to those durability factors discussed above, the following differences in durability between 2BMA trial mix and the suggested ternary mix are also apparent.

For sulfate resistance, concretes containing up to 40% replacement of limestone fines showed similar performance to CEM I concretes. Increasing the calcium content in concrete also increases the risk of thaumasite type of sulfate attack. Concretes containing GGBS replacements show superior performance compared with all other cement types.

Acid resistance for GGBS concrete is increased, whereas concretes containing limestone fines are more susceptible due to the increased calcium carbonate content.

For alkali silica resistance, GGBS is accepted worldwide as a means of reducing the risk of damage due to ASR (Bakker, 1981, Higgins and McLennan, 2009). There have been many investigations, and these have generally confirmed the ability of GGBS to reduce significantly the deleterious expansion caused by ASR, especially when used to partly replace high-alkali CEM I. It is reported that no concrete structures with greater than 35% blast furnace slag content have suffered damage from ASR in practice. The mitigating mechanism is not completely understood, or fully quantified, but the low permeability of GGBS concrete to alkali ions is probably an important factor, as well as the slower release of alkali metals from the GGBS (Hobbs, 1986).

Hobbs (1983) reported that the use of 5% limestone extended the time to, but did not eliminate cracking in mortar bars made with high-alkali cement.

4.2.5. Concrete Batching

Batching of the 2BMA concrete occurred using different methods in each of the two sets of trials. In the first trials, concrete was batched at a dedicated batching plant some distance away from the trials site, and in the second trials, small scale mobile batching and mixing plant was utilised. This second method causes problems with batch consistency, as the concrete is mixed in tumble mixers instead under higher shear of a mixing pan with forced action paddles.

Despite its proximity to the mouth of the tunnel, the batching of the concrete using mobile plant was time consuming. Batching times are considered to be very slow, at around 14 m^3/hr. This aspect can hinder the transport, placement and compaction of the concrete, increasing the risk of quality issues.

Regardless of the method, issues with the fresh properties of the concrete were apparent as already discussed. On balance, it would always be better to batch using a dedicated high volume modern plant as the accuracy and speed of batching would be much improved.

The final set up of batching facilities will be dependent on the sequence of caisson construction. For example, if all caissons are constructed together, then it would be prudent to invest in high quality mobile wet mix batching plant solutions, using computerised weigh batching, effective moisture correction and force action mixing of concrete.

4.2.6. Use of Admixtures

The issues with the 2BMA mix in relation to workability retention and set retardation could be effectively fixed through the combination of optimised admixture solutions. Despite this, it is considered on balance, that GGBS ternary blend concrete be considered, due to naturally slower setting times, lower heat production and improved durability.

Nevertheless, powerful set retarding admixtures are available that allow long term set retardation of concrete (i.e. over 24 hours). These can be combined with accelerators to “reactivate” the concrete when required. Combinations of polycarboxylate-based superplasticisers allow high ranges of workability that ease pumpability and placement, along with workability retention.

Further discussion with admixture suppliers to fully appreciate the options should be carried out.

4.2.7. Conclusions Regarding Concrete Mix Design

Despite attaining favourable hardened properties, the concrete used for the 2BMA caisson trials consistently reported issues with rheology and fresh properties, resulting in significant issues in consistent placement of the concrete. The constructability risk of the proposed mix is questioned, with the principal reason behind the problems being excessive proportioning of limestone fines in the concrete mix. There is also the potential risk of early thermal cracking.

According to the information available for this review, SKB has not adequately justified the use of the cement mix with high concentrations of limestone. It is not clear whether they have considered using a ternary cementitious blend of concrete containing CEM I, GGBS and smaller proportions of limestone fines. Test results reported and experience indicates that such a mix has the potential to have fewer issues during placement, with similar hardened mechanical properties and potentially improved durability.

It is recommended that mixes such as this are considered further.

4.3. Cracking Risk

4.3.1. General

During the 2BMA trials, monitoring for cracking was carried out, and it was reported that no cracking was observed during monitoring of any of the trial works. In most cases no cracking would be expected, such as where in the first set of trials (Mårtensson and Vogt, 2019) in casting of Section 1 (Component 1) the base and walls were cast integrally. Similarly in the second set of trials (Mårtensson and Vogt, 2020) the base slab was preheated (i.e. pre-expanded) prior to casting of the walls, thus reducing restraint during hydration of the walls, as the base and walls would contract together during cooling.

Section 2 (component 2) in the first set of trials may have been expected to show cracking, as the wall was cast on a stiff base, although this would have been dependent on a number of factors, principally the concrete placement temperatures.

This section explores the risk of cracking for caisson construction, first by looking at simple restraint of the elements and then at the thermal behaviour of the concretes.

4.3.2. Restraint

Types of restraint and the effect on crack widths

Early age thermal cracking is the result of restraint to the thermal contraction which occurs as the concrete cools from its peak curing temperature back to ambient conditions. The restraint can develop either within a pour or between pours. Restraint with-in a pour occurs when one part, typically the centre, heats up and cools down to a greater extent than the surface through which heat is lost. This type of restraint is defined as internal restraint.

Restraint between pours occurs when a new pour is cast against or between existing concrete elements and the thermal strain in the new pour is restrained by the stiffer element against which it is cast. This type of restraint is defined as external restraint and can occur in two principal forms.

Continuous edge restraint occurs when adjacent pours are cast. The most common form of edge restraint develops when a wall is cast onto a stiff foundation, but this also develops when adjacent pours are cast in the same plane, i.e., adjacent slab pours. In this case the restraint develops along the direction of the joint.

End restraint develops when a pour is cast between existing stiff elements. The most common form of edge restraint develops when a slab is cast between local restraints such as core walls or columns or when infill pours are cast. In this case, the restraint develops perpendicular to the joint.

These general forms of external restraint are illustrated in Figure 23.

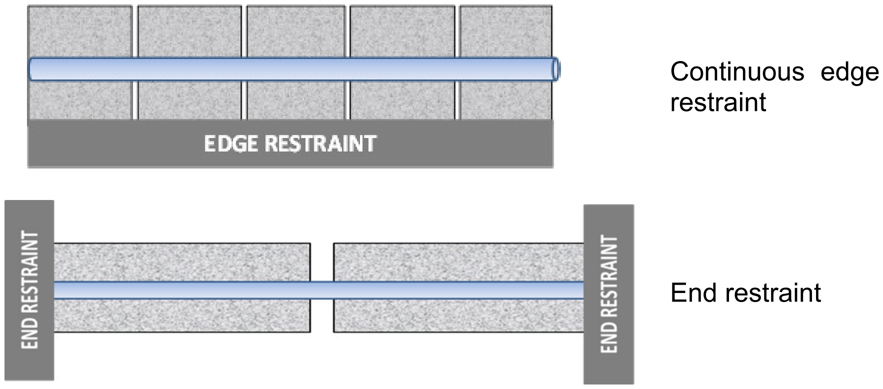

Figure 23: Forms of external restraint (Bamforth, 2018).

Cracking will occur when the restrained tensile strain in the new element exceeds the tensile strain capacity of the concrete. However, for a given amount of reinforcement, cracks tend to be much larger when caused by end restraint. This is because the reinforcement alone is relied upon to limit the crack width. Under conditions of edge restraint, some of the load in the concrete prior to cracking is transferred to the adjacent restraining element when a crack occurs, and less of the load is therefore transferred to the reinforcement. With lower stress in the steel the crack widths are smaller.

4.3.3. The Caisson Walls

A caisson wall is an example of a member with continuous edge restraint, in this case provided by the 600 mm thick base slab onto which it is cast. Edge restraint is zero at free ends of a pour and builds up progressively with the distance from the free end. EN1992-1-1, §7.3.4 provides guidance on the crack spacing when a wall restrained by a previously cast base is under-reinforced, and recommends a value of 1.3 x height (H). If the wall is cast onto the previously cast slab, then it may be assumed that the maximum restraint develops at a distance of 1.3 H from the free end (Figure 24).

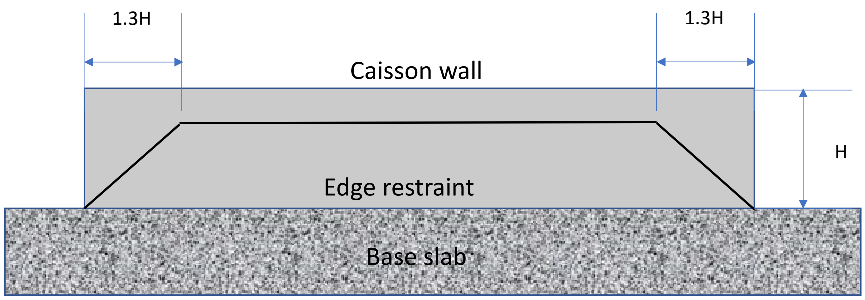

Figure 24: Longitudinal edge restraint profile (adapted from Bamforth (2018))

Hence to avoid cracking, the length of a member should be less than 2.6 H. The caisson wall is 9m high. To avoid cracking due to edge restraint the length of each pour would have to be less than about 23.4m. Once this length is exceeded, the crack width is independent of the length and no limit on the length need be applied. As the walls of the caissons in the reference design are 18.1m in length, cracking due to external edge restraint is not considered likely.

Despite the apparent low risk of cracking, restraints have been calculated in accordance with the guidance of CIRIA C766 (Bamforth, 2018).

4.3.4. General Layout of the Caisson Walls

The caisson walls form the perimeter of the base slab and are cast in 18.1m by 18.1m lengths.

4.3.5. Calculation of Edge Restraint

Edge restraint imposed on a pour at the joint with existing concrete calculated using equation (1).

$$R = \frac{1}{\left(1 + \left(\frac{A_n E_n}{A_o E_o}\right)\right)} \qquad (1)$$

Where: A_n is the csa of the new pour
A_o is the csa of the old pour
E_n is the elastic modulus of the new pour
E_o is the elastic modulus of the old pour

According to CIRIA C766 (Bamforth, 2018), when a wall is cast at the edge of a slab, the relative area should be assumed to be the same as the relative thicknesses. As the base slab is 600mm thick and the capping beam is 680mm thick, A_n / A_o =1.13.

Bamforth (2018) recommends that the ratio $E_n / E_o = 0.7$ for thick sections which cool relatively quickly.

Hence:

$$R = \frac{1}{(1 + (1{,}13\ x\ 0.7))} = 0.56$$

The variation in the restraint with height, for the 18.1 m length of wall is shown in Figure 25. The maximum restrained strain does not occur at the joint, but at a height above the base equal to 10% of the length. For the 18.1 m length of wall, the maximum restrained strain occurs at around 1.8 m from the base of the wall, also shown in Figure 25. At this location the restraint has reduced to 0.42.

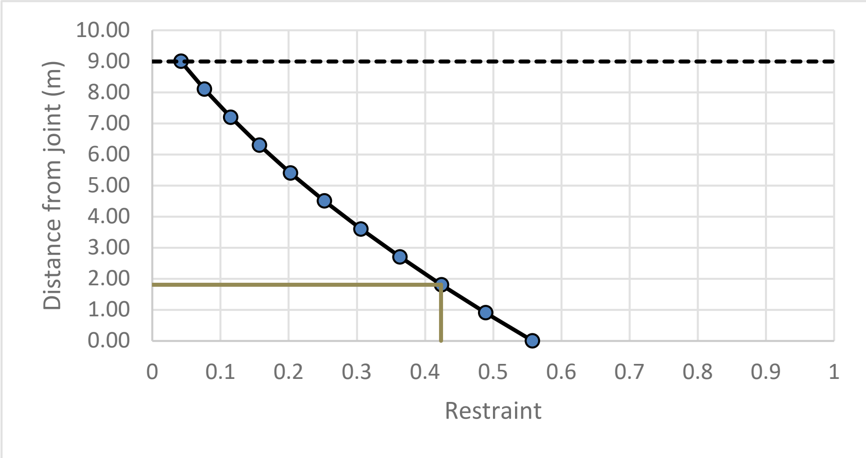

Figure 25: Variation of restraint with height of caisson wall.

4.3.6. Thermal Behaviour

Both the 2BMA concrete and the suggested ternary blended concrete were compared in terms of their likely thermal behaviour.

The suggested ternary blended concrete mix is considered to behave in a beneficial manner in relation to the risk to early age thermal cracking, and in fact potentially would show improved thermal behaviour over the current 2BMA mix tested by SKB. This is demonstrated through the use of empirical model comparisons between the 2BMA mix and suggested alternative.

First, the adiabatic (heavily insulated) heat rise data from the suggested ternary mix (see Section 4.2.5) was calibrated against the parameters used in the empirical finite difference model given in CIRIA Report C766. Figure 26 shows the calibration of the measured test data against the modelled heat development curve. This allows for the use of accurate model parameters during model comparisons.

As no thermal data is available for the 2BMA mix, the CIRIA C766 model (Bamforth, 2018) was used to estimate the in-situ temperature of the actual wall temperature monitoring data.

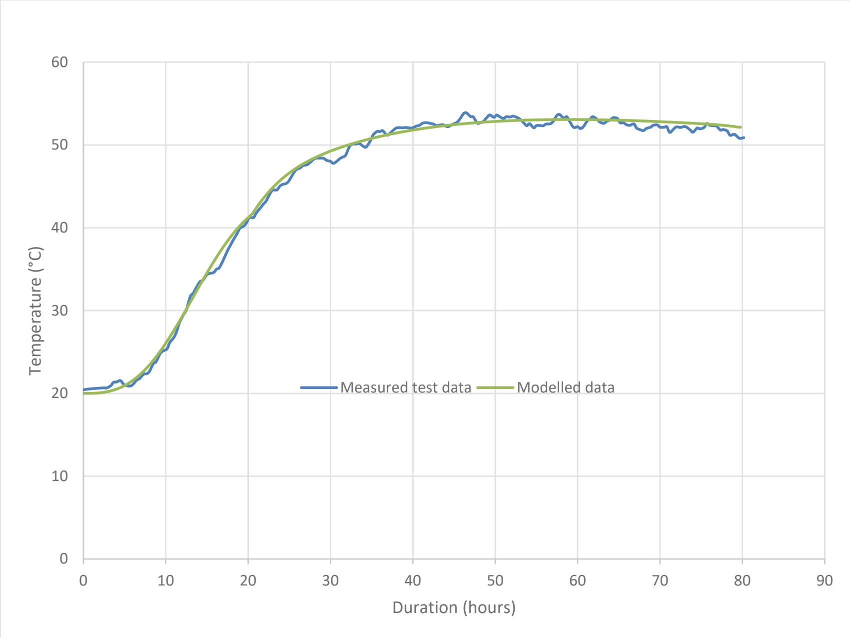

Figure 26: Calibration curve for the suggested GGBS ternary blend concrete against CIRIA C766 finite difference model (Bamforth, 2018).

The 2BMA mix was then assessed within the CIRIA model. The wall cast monitored during the second set of trials was selected for comparison, with a wall thickness of 680 mm. As the limestone fines are not taken into account in relation to any heat generation, the model assumes a binder content of the CEM I only, at 320 kg/m^3. The following parameters are assumed for the comparison between the two concretes:

- Section thickness: 680mm
- Surface heat transfer coefficient (18mm plywood at 0m/s wind speed), 3.08 W/m^{2}k
- Formwork removal, 72 hours
- Thermal conductivity, 1.85 W/m °C
- Concrete placement temperature, 10°C (8-12°C reported)
- Ambient temperature (placement temperature minus 5°C), 10°C
- Thermal expansion coefficient (Basalt aggregate), 10 microstrains / °C
- Compressive strength, C40/50
- Seasonal temperature change, 0°C (elements expand and contract together)
- Mean relative humidity, 65%
- Long-term shrinkage, 290 microstrains.

The outcome of the model comparison is shown in Table 9 below.

Table 9: Results of comparison modelling between 2BMA and ternary mixes.

	Peak temperature (°C)	Mean temperature drop (Tm) (°C)	Temperature limit to cause cracking	Risk of cracking
2BMA mix (320kg/m^3 100% CEM I)	43 @ 40 hours	31.0	21.8	Yes
GGBS ternary mix (390kg/m^3 45%/55% CEM I / GGBS)	33 @ 51 hours	21.6	21.8	No

The CIRIA model accurately predicts the peak temperature measured in the 680 mm thick walls of the trial cast, at 43°C (Mårtensson and Vogt, 2020, Section 11). Using the same boundary parameters as the 2BMA mix, the GGBS ternary mix estimated results are compared in Table 9 above. This table shows that the ternary mix will generate less heat during hydration, thus reducing further any cracking risk.

The table also shows the limit of the mean temperature drop required to avoid cracking (strains are induced as the concrete cools against the restraint from the base slab). Assuming the temperature across the section is parabolic, with peak temperature T_1 and surface temperature T_s, the mean temperature can be estimated using the equation:

$$T_m = T_1 - \frac{(T_1 - T_s)}{3} \tag{2}$$

Assuming the restraint between the slab and wall (Section 4.3.5) the limiting temperature drop to avoid cracking is calculated as:

$$T_m = \frac{\varepsilon_{ctu}}{K_{c1}\alpha_c R_1} - \frac{\varepsilon_{ca}}{\alpha c} \tag{3}$$

Where: ε_{ctu} is tensile strain capacity of concrete (at early age)
K_{c1} is a factor allowing for creep on stress relaxation at early age (0.65)
α_c is thermal expansion coefficient of concrete
R1 is restraint (0.42)
ε_{ca} is autogenous shrinkage strain

Table 9 shows that the mean temperature drop of the 2BMA trial concrete is great enough to exceed the risk of cracking, whereas the suggested ternary mix is just beneath the limiting temperature. This can be improved further by increasing the replacement level of the GGBS.

The fact that no cracking was seen during the trials, particularly where expected, does not mean that it would not occur during future construction events, particularly if the placement temperature of the concrete is increased (for example, during summer casting conditions). Therefore, the implementation of a lower risk mix is considered prudent.

4.3.7. Conclusions Regarding Cracking Risk

In conclusion, the suggested ternary mix indicates improved thermal properties to that of the selected 2BMA mix. Although the wall-on-slab geometries indicate low crack risk, areas of possible higher restraint may exist. If areas of higher crack risk are considered possible, the improved thermal properties of the ternary mix may have an advantage over the selected 2BMA mix design in relation to thermal behaviour and overall crack control, along with improvement in other properties mentioned in previous sections.

Further discussion on crack control is provided in Section 4.6.

4.4. Joint Design review and risk

4.4.1. General

Both trials reports made an effort to assess the permeability of the joint between the base slab and the walls of the caissons. In the first report (Mårtensson and Vogt, 2019), the joint layout examined was that shown in Figure 3 (repeated below in Figure 27) where one wall was cast integrally with the slab, and one wall was cast sequentially with either one or two copper plate waterstops.

The main reason for casting the base and walls integrally is the formation of a joint with possibly poor hydraulic properties when casting the slab and wall separately.

It was concluded in the first report that the hydraulic conductivity of the joints and the interface between the joint seal and the concrete was found to be closely similar to that of the bulk concrete, therefore stating separate casting of slabs and walls is possible without joint seals. This is considered questionable.

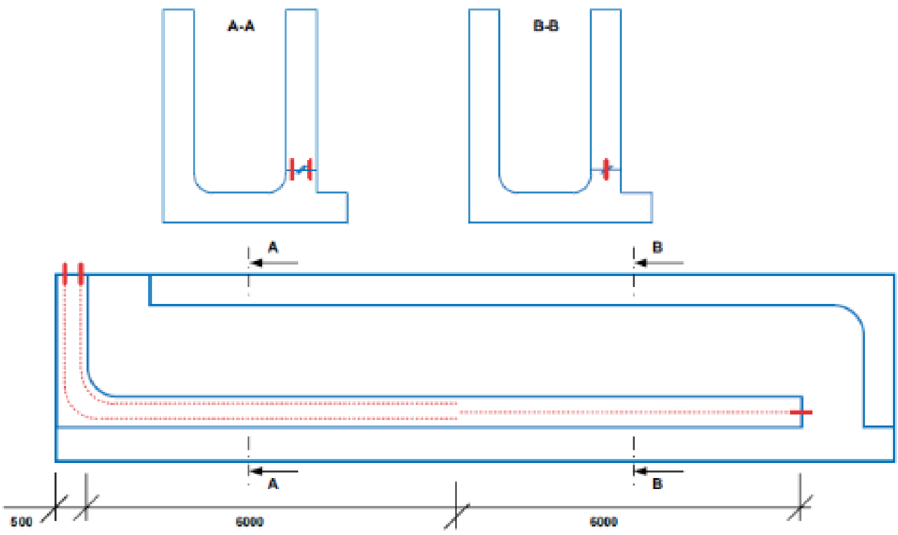

Figure 27: Layout of joints in the first set of trials (SKB TR-18-12).

The integral casting of the slab and walls was reported to be problematic, resulting in poor surface finish of the base slab. A re-evaluation of the method, and additional measures would be required if integral casting remains the preferred approach in the actual construction of the caissons. On the basis of the information provided in the current review, this method is considered a risk to consistent quality of caisson construction. Therefore, a more reliable procedure would be to cast the slab then the walls and control the other aspects of the construction risk, by improving joint quality and considering second level protection.

In the case that the walls are cast separate to the slab, the reports state that careful preparation of the joint surface prior to casting would improve the hydraulic properties of the joint.

4.4.2. Joint Quality and Hydraulic Conductivity

After casting in the first trials, the joint seals were examined and reported to be good, but could have been better due to the problems with the concrete, such as poor aggregate distribution at the interfaces between walls and slab (Figure 28). This report again highlights the risk of poor joint connections with the concrete used.

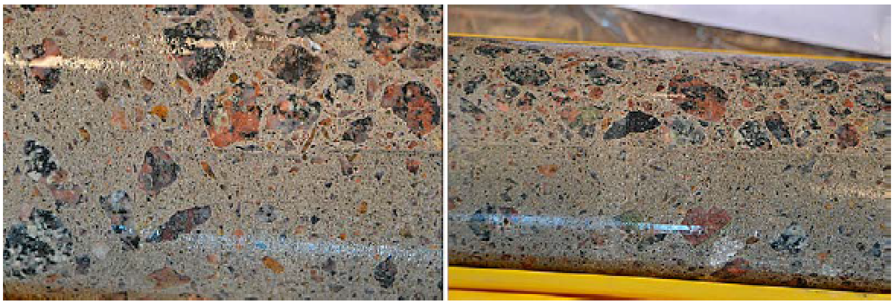

Figure 28: Poor aggregate distribution at wall and slab interface (SKB TR-18-12).

Examination of the adhesion of the joint seal using the copper plate was not really possible as the core split at the plates. Hydraulic conductivity testing was attempted, but the method of testing was not sufficient to accurately assess the hydraulic conductivity at the joint (only that it was less than 1×10^{-11} m/s). This value is an order of magnitude higher than would normally be recorded for good quality concrete, and therefore does not prove anything.

In the second report (Mårtensson and Vogt, 2020), some minor issues were seen with the joint between the base slab and the walls, where the concrete was not compacted completely.

In the second set of trials, hydraulic conductivity of the joint was assessed, as has been previously reported above, and repeated below (Table 10).

Table 10: Hydraulic conductivity (water permeability) coefficients for the 2BMA trials.

Specimen	Hydr. cond. [m/s]	Direction of drilling	Comment
F08000008a	5.4×10^{-8}	Across the core	In the joint
F08000008b	3×10^{-11}	Across the core	In the joint
F08000011 IN	1×10^{-11}	Along the core	Centre of wall
F08000013	3×10^{-12}	Along the core	Centre of wall

The conclusion of these second trials was reported to confirm previous findings from the first trials. The average hydraulic conductivity for the bulk concrete was 6.5×10^{-12} m/s, whereas it was 2.7×10^{-8} m/s for the joint, a difference of over four orders of magnitude. Even removing the high joint result and relying on a single value still results in a difference of an order of magnitude.

It is the opinion of the reviewer that the assertion in the reports stating there is no difference in hydraulic properties between a well-prepared joint and a copper sealed joint has not been proven.

Moreover, it is considered a high risk to assume that joint preparation alone would be sufficient to provide low hydraulic conductivity for the structure. Preparation of the joint between the slab and the wall alone cannot necessarily be relied upon to ensure the high-quality waterproofing requirements at the joint. Despite the fact that these reports represent trials, certain quality issues during construction activities are highlighted, indicating there are construction risks to quality assurance inherent in the process. If joint preparation alone is to be relied upon to provide waterproofing, SKB should provide more evidence that it will meet the required performance. If, as is likely, this cannot be done, then joint preparation alone should not form part of the final design; additional waterproofing measures will be required at critical joints.

4.4.3. Options for Joint and Joint Waterproofing

It is considered that casting the base slab and the walls integrally introduces risks to construction quality. Given the space constraints within the 2BMA cavern separate casting of walls and slabs would be more appropriate.

As stated above, for waterproofing structures, it is not considered best practice to rely solely on joint preparation for waterproofing due to quality assurance risks during preparation and construction of the walls.

There are a range of solutions that could be considered for waterproofing protection of the joints. It is also generally good practice to supply two lines of defence within a waterproofed structure, in case one system is compromised.

In terms of durability of the waterproofing system, this has to be taken in context with the waterproofing capability of the concrete itself. For example, scoping calculations suggest that a high-quality concrete with a hydraulic conductivity of 5×10^{-13} m/s would be penetrated by water at a hydraulic head of 150m at around 80 years. In this context, it is considered joint waterproofing systems are not necessary beyond this timeframe. This potentially allows consideration of the introduction of materials that are durable at engineering timescales.

The following options are provided for information. A more detailed assessment would need to be made on the specific merits on the systems or combinations.

Waterstops/waterbars

Waterstops are physical barriers that cover the joint in various ways. For example, the copper plate used in the trials would be regarded as a waterstop. Waterstops can provide improvements in direct barrier protection, movement provision and tortuosity.

Selection of material type (i.e. synthetic rubber, HDPE, polyesters, resistant alloys etc) would need to ensure resistance to alkalis over the long term, along with resistance to chlorides. Figure 29 below shows design configurations for both external and internal waterstops.

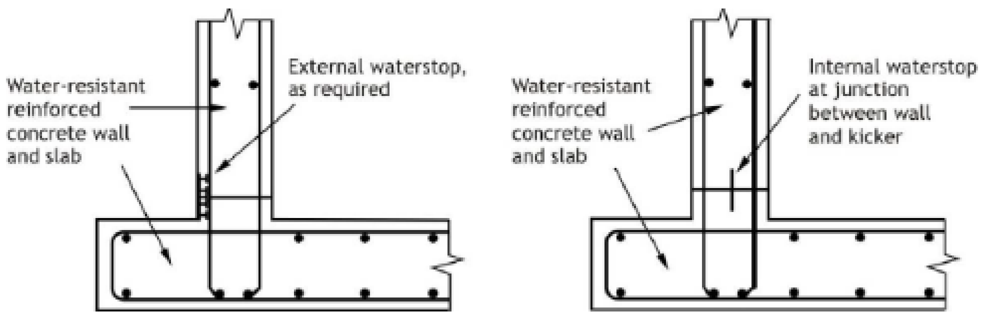

Figure 29: Example options for the use of waterstops (internal or external).

Waterbars also provide a physical barrier to ingress due to their swelling properties in the presence of water. Water bars are made of hydrophilic materials that expand in the presence of water (i.e. at a leaking joint), thus physically cutting the penetration of water (Figure 30, Figure 31).

Selection of materials should include swelling properties in the presence of chloride-bearing waters.

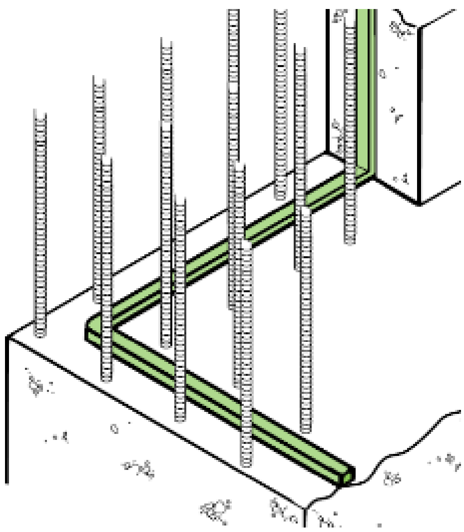

Figure 30: Typical layout of hydrophilic waterbar at joint.

Figure 31: Hydrophilic waterstop in position on top of an integral kicker.

Integral waterproofers

Integral waterproofers are added to the concrete as admixtures and can produce concretes with comparatively low hydraulic conductivities. Their mechanisms range from pore blocking actions by the promotion of crystalline growth, to pore lining with hydro-phobic polymers.

Tests have shown that concretes with a water resisting admixture provided an improvement in test performance over their respective control concretes. However, some of the control concretes performed better in the tests than some of the concretes with water resisting admixtures, suggesting that an integral waterproofer is not necessarily required in order to achieve a concrete with water-resisting properties.

Re-injectable hose

Hoses may be placed within the joint and cast in place. Typically, injection materials are pumped through the hoses if water leakage is observed, although it is possible to inject during post construction regardless of leakage.

A range of materials may be injected, from acrylic, polyurethane or polyester resins, to microfine cementitious grouts (e.g. Figure 32).

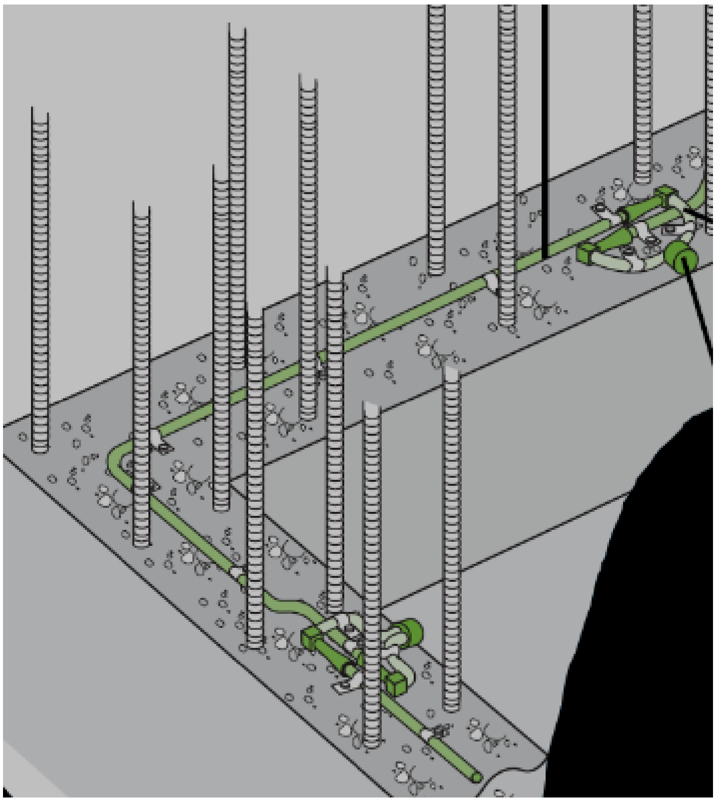

Figure 32: Example of embedded reinjectable hose waterproofing.

Membranes

Surface applied membrane types may be considered as part of a joint waterproofing system (Figure 33).

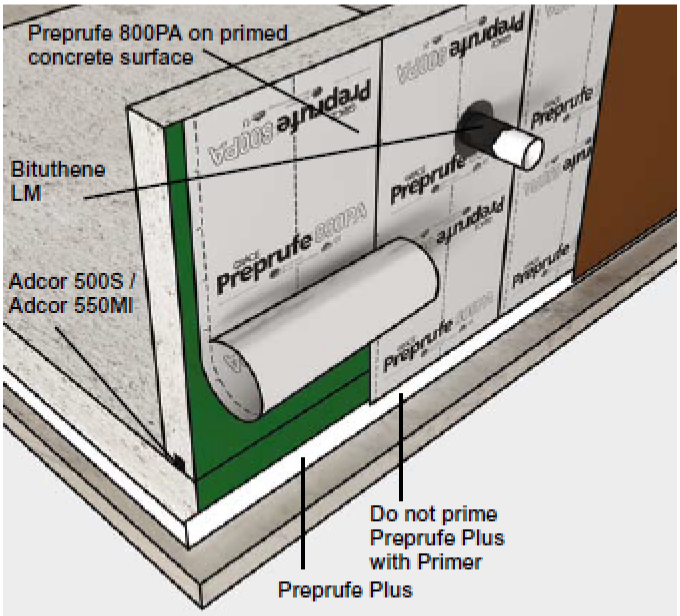

Figure 33: Example of higher quality post applied membrane system for use on joints.

Surface applied membrane sheet systems are numerous and range from low-grade tanking systems using bituminous-based products to more sophisticated membranes using HDPE and Hypalon.

Simple tanking systems typically comprise self-adhesive-backed bituminous- or HDPE-based sheet membranes. These are simple post-applied systems used for waterproofing of substructures and basements, providing gas and water resistance, and are a cost-effective option for basic waterproofing of shallow structures. However, most of these sheet waterproofing materials are not tolerant to moist or damp substrates, although some products are available that utilise moisture-tolerant primers for application on damp (not wet) substrates.

Most these materials are aimed at the market for shallow basements and substructures. The waterproof testing of these materials is limited typically to testing of up to 6 m head, so are not generally applicable to deep basement structures, which is a key reason to discount their use.

However, a small range of membrane sheets do have test data demonstrating resistance to high hydrostatic heads and are also moisture tolerant. Bituthene 8000 is a bituminous based sheet material and Preprufe 800 PA are materials from Grace Construction that provide good levels of water resistance and chemical resistance to ground conditions, although they must only be used on the positive side of walls, which means they cannot be used in the inner wall surfaces of the vault. The adhesives and associated protection boards also have a high VOC content and are extremely flammable.

One major disadvantage of these materials is that the adhesive has a limited life span in open air, particularly on damp substrates, as moisture trapped behind the adhesive causes blistering of the membrane. Typically, these materials are not to be exposed for more than 30 days, and therefore walls should be backfilled as soon as possible.

Despite some advantages in ease of application and cost effectiveness, these materials are not considered appropriate for walls that will be left open to the ambient climate for extended periods of time, as their performance may be impaired.

Higher grade products are also available that are more specifically designed to seal movement joints and cracks in concrete structures (Figure 34). These materials typically comprise a flexible strip of elastomeric waterproof sheet and epoxy-based adhesive. These materials are typically used to waterproof expansion joints and live cracks, having high movement accommodation. These strip membrane systems are also primarily developed with joints in mind, whose linear orientation is typically planar, normal to the concrete surface.

The disadvantages of these materials are that they are relatively expensive (over twice the cost per linear metre compared with liquid-applied systems) and are heavily reliant on good workmanship to ensure adequate adhesion to the substrate and subsequent performance.

Other issues concerning these systems are that careful consideration is required when selecting the primers and adhesives, many of which are not moisture tolerant.

Figure 34: Example design and installation of high-quality joint waterproofing.

Coatings

Both organic and inorganic coatings are commonly used on concrete structures. Typically, high build organic compounds, such as glass-flake or micaceous epoxies, are used to protect against aggressive chemicals. Their principal use is corrosion protection of steel substrates where there is a long and established track record. There is very little track record for these materials on concrete substrates used in structural waterproofing applications, and so these are not considered further.

Cementitious coatings, modified with organic polymers are increasingly used to protect against water ingress and chemical attack (Figure 35). Most systems are brittle and un-suitable for active cracks. A range of flexible polymer-modified cementitious coatings are available for use for structural waterproofing of concrete over cracks and have a long and established track record.

The advantage of using coatings for waterproofing repairs is that the quality of both the coating material and the workmanship in applying it can be readily observed and recorded. This allows any remedial works to be easily verified. The process of application of the coating is also more straightforward than other methods, requiring only a good surface preparation of the concrete. Workmanship is key factor in the success of a coating to provide its intended protection, so therefore a robust QA and supervision process is essential.

Figure 35: Application of high-quality cementitious coating on horizontal construction joint.

4.5. Tie Hole Review

4.5.1. General

The pressure exerted by the fresh concrete during placement requires structural strengthening of formwork systems to allow them to remain in dimensional stability. Formwork ties are used to hold formwork together during the process of concreting most vertical units. Form ties are necessary in most types of concrete construction, although alternatives exist where they are appropriate.

4.5.2. Alternative Formwork Systems

Systems providing an alternative to tie rod formwork typically fall into two categories. The first would be the single-sided/propped formwork system, whereby formwork pressures are resisted by the support frame which is bolted to a sturdy foundation (Figure 36). The alternative would be to brace the formwork between walls (either as constructed or within cavern walls).

Figure 36: Example of self-bracing formwork.

In addition, tie-less concrete construction can be carried out using a slipform system. Slipform concrete processes is a specialist method of construction of vertical concrete structures, or the production of horizontal paving structures (e.g. slabs, barriers etc.), and is one of the fastest methods of concrete construction. The concrete is effectively extruded from the moving formwork.

For vertical structures, the process utilizes a self-climbing formwork system comprising of shuttering that straddle the structural element and are supported by yoke lifted by a series of mechanical jacks. The jacks are supported by rods or tubes positioned within the wall or structure which are laterally braced by the rising concrete structure. Rates of rise of up to 450mm per hour can be achieved with good practice and management. The process is typically used for high-rise structures, but a small slipform unit could be considered for the 2BMA vaults.

The disadvantages of these systems are potentially the space requirements within the vaults, along with bespoke design and construction cost requirements.

Tie rod systems for wall construction appear to be a more preferable option in terms of constructability.

4.5.3. Formwork Tie Systems

There are various types of tie that range from light to heavy duty, and those that can be removed and reused. Those used for more heavyweight purposes are systems such as threaded tie, taper tie and she-bolt tie systems, shown in Figure 37, Figure 38 and Figure 39 with simple description, all of which can be removed from the constructed element and the tie hole remediated.

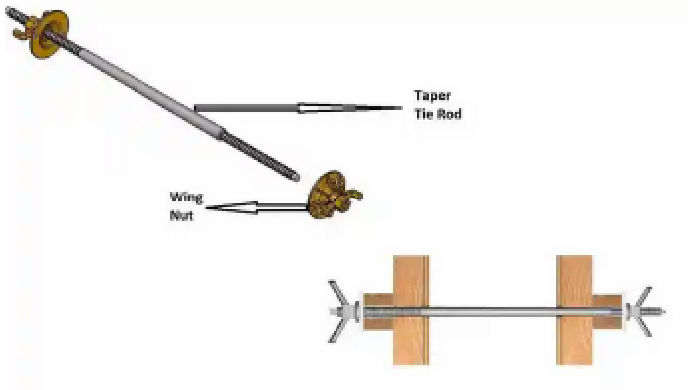

Figure 37: The taper-tie system. Taper rod system is threaded at both ends, locked by wingnuts and washers. Ties can be removed completely but must be coated in grease/ surface retarder before installation.

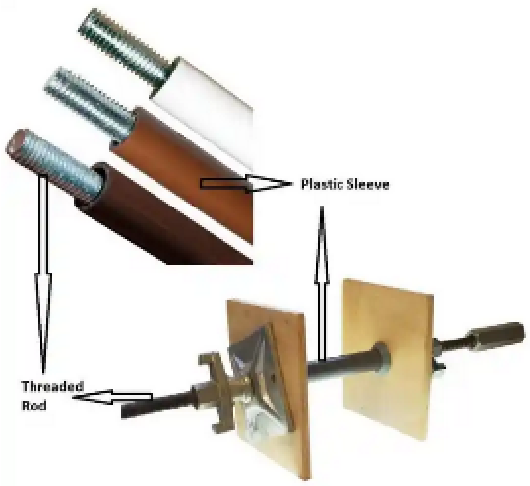

Figure 38: Threaded tie system. The threaded tie system is a fully threaded bar located within a plastic sleeve. The threaded tie can be removed after concreting and the plastic sleeve remains in place.

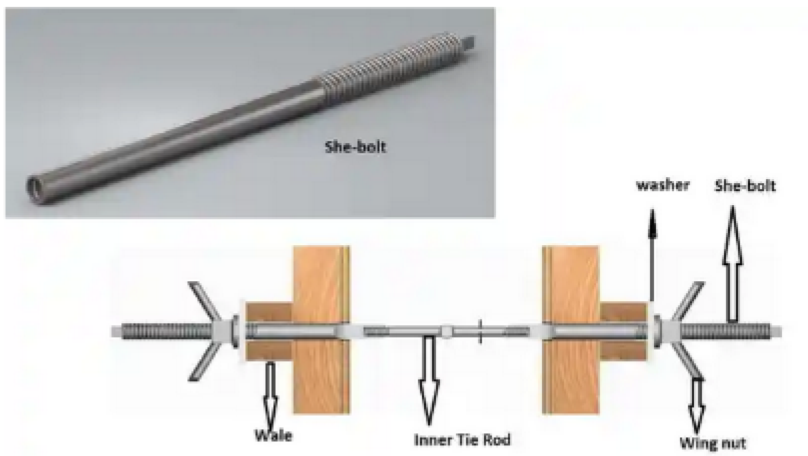

Figure 39: She-bolt tie system. The she-bolt tie system contains one inner tie rod and two she-bolt members that are used to tighten and fix the formwork. After concreting, the inner tie rod remains in the concrete and the she-rods can be removed.

Each of the above systems have their merits, with the latter two systems leaving residual items within the concrete. The plastic sleeve may cause particular issues in relation to permeability and may need to be removed (e.g. by heat treatment) but this removal may be problematic in reality. The inner rod of the she-bolt system is probably small enough not to cause significant corrosion issues and could also be made more resilient by using non-ferrous materials.

There are also systems on the market that allow alternative closure of tie holes. One system that is considered worth investigating is the DK, SK tie system technology from Peri. In summary, the system is similar to the threaded tie system described above, but with a bespoke closure system for the tie holes. Selected images are provided below (Figure 40). Further discussion is recommended.

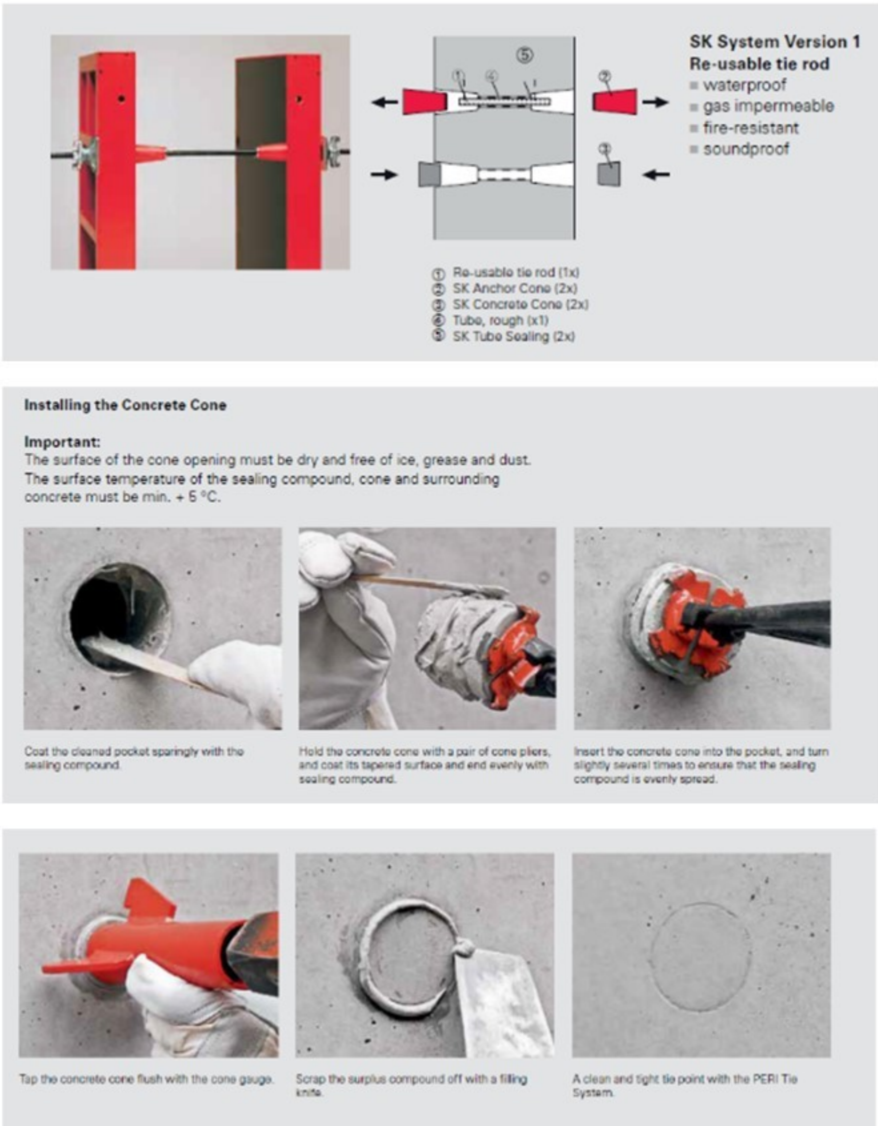

Figure 40: Application of tie rod removal and repair system (Peri®, 2019).

In conclusion, more advantageous systems for the application are likely to be available than the tie-rod systems that were used for the trials.

Filling of the holes also allows some flexibility. The second stage trials report the attempted use of a rudimentary pumped system for tie hole filling, with various degrees of success. One or more of the following possible options may provide better results:

- Expanding polyurethane or acrylic grouts.
- Post hardening expansive cementitious grouts (e.g. calcium sulfo-aluminate grouts).
- Pressure injected cementitious grouting procedures.

4.6. Reinforcement

4.6.1. General

The project requirements state that no steel reinforcement should be used in the design of the caissons. This is understandable, as ongoing corrosion would produce expansive forces and potential cracking of the concrete, leading to an increase in hydraulic conductivity.

The caissons have been designed so that no structural reinforcement is required. However, it is considered that reinforcement to control early thermal cracking and drying shrinkage may be necessary, depending on the concrete used, the geometry and environmental factors. To this end, a brief review has been carried out to describe some of the options available for the use of non-ferrous reinforcement. The list is not exhaustive, but does represent the majority of materials available for use.

4.6.2. Reinforcement Materials

Most material types for non-ferrous reinforcement are based on the approach using particular fibre types that are encased in a resinous matrix. Common to most of these materials is the fact that the reinforcing typically has higher tensile strength but lower stiffness when compared with more conventional carbon steels. The mechanical behaviour of fibre-reinforced plastic (FRP) reinforcement differs from that of conventional steel reinforcement. FRP materials are anisotropic in nature, characterised by high ten-sile strength in only one direction but with low ductility. They are elastic until failure. Their use as crack control reinforcement should therefore be generally well suited.

Typical fibre types are:

- Fibre reinforced plastic (FRP/GFRP)
- Aramid fibre composite (AFRP)
- Carbon fibre composite reinforcement (CFRP)
- Basalt fibre reinforcement (BFRP)

FRP properties are summarised in Table 11. Density ranges from 1.25 – 2.1 kg/m^3, around one quarter to one sixth of that of conventional steel. This means transportation and placement is much easier and safer.

Table 11: Summary of FRP reinforcement properties.

	Steel	GFRP	CFRP	AFRP	BFRP
Nominal yield (MPa)	279-520	-	-	-	-
Tensile strength (MPa)	480-1600	480-690	600-3690	1720-2540	~1400
Elastic Modulus (GPa)	200	35-51	120-580	41-125	~55

Composite fibre reinforcement is typically created by pultrusion. The pultrusion process creates fibre/resin matrix composite rods (up to 30m in length depending on the supplier). Resins used are typically vinyl esters, but epoxy and phenolic resins are also available that offer greater resistance. The rods are spiral machined or twisted to create a deformed-type profile for greater concrete bond. FRP reinforcement is very easy to cut to size. Pre-bent bars can be ordered in accordance with design requirements. Figure 41, Figure 42 and Figure 43 illustrate the typical process.

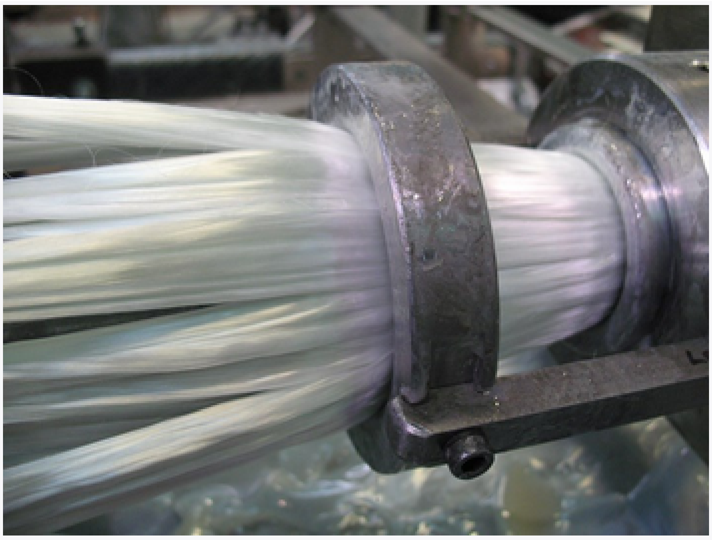

Figure 41: Fibre passing into pultrusion die.

Figure 42: Fibre pultruded as reinforcement rods.

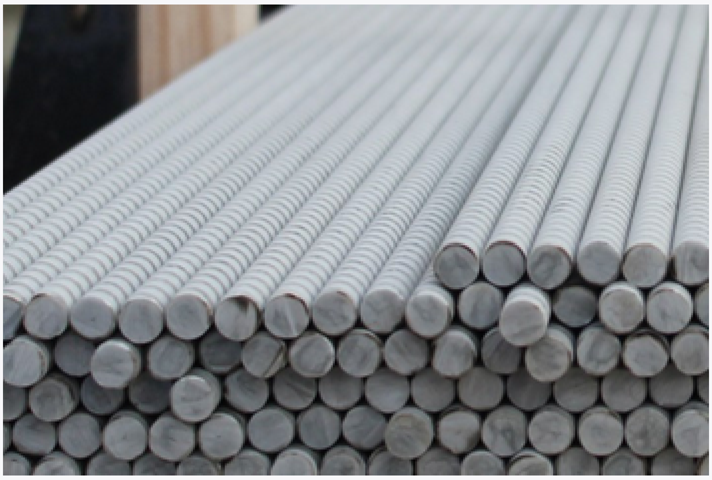

Figure 43: Finished ribbed fibre reinforcement.

4.6.3. Limiting Crack Width

As the caissons are designed with no structural reinforcement, FRP reinforcement would be required only to control early thermal cracking. CIRIA C766 (Bamforth, 2018) provides guidelines on how to design for effective crack control.

For water resisting structures, guidance on the limiting crack width is based on self-healing of the concrete at a specific hydraulic gradient, in accordance with EN 1992-3 (EN 1992-3, 2006).

When cracking is caused by external restraint to early thermal contraction it is likely that a crack with a surface width of 0.2 mm will be considerably wider in the core of a section, where the temperature rise and fall was greatest. The research on autogenous healing has generally been on specimens cracked under load and of uniform width and the proportion of the crack which needs to be 0.2 mm in width for self-sealing is not known.

It is generally assumed that where cracks are dynamic, self-healing is unlikely to occur. Even if the provisions for tightness class 1 are met EN1993-2 draws attention to the fact that cracks through which water flows may be expected to heal in members which are not subject to significant changes of loading or temperature during service. EN1992-3 states that “If self-healing is unlikely to occur, any crack which passes through the full thickness of the section may lead to leakage, regardless of crack width”.

Research by Edvardsen (1999) however, has investigated the effect of crack movement and demonstrated that self-healing may occur in narrower cracks even if some movement occurs. Testing was undertaken in which cracks were subjected to either 10% or 30% increase in crack width on a one-day cycle. Based on these tests, Edvardsen (1999) proposed limiting crack widths based on the expected crack movement such that the cracks will be self-healing within a few weeks. These results are shown in Figure 44 compared with the recommendations of EN1992-3. The suggested limits given in Figure 44 show where it can be demonstrated that the crack movement will be ≤ 10%.

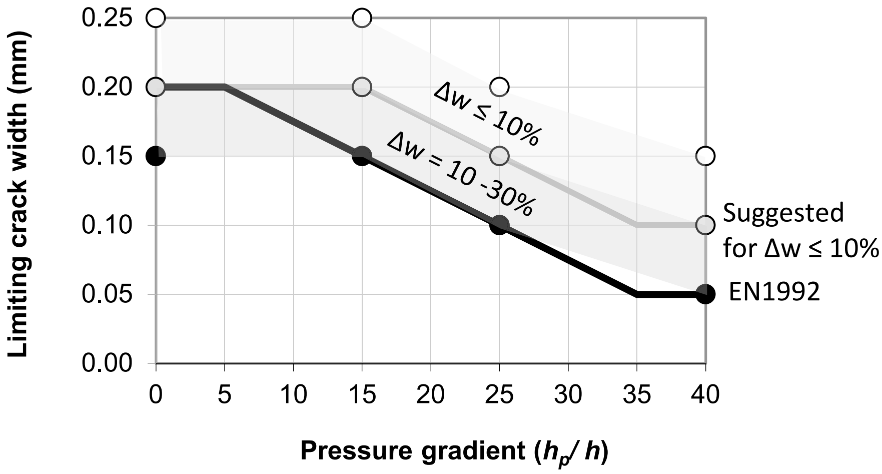

Figure 44: Limiting crack width for self-healing within a few weeks according to the crack movement (recommendations of Edvardsen (1999) shown as symbols) (Bamforth, 2018).

Regardless, 2BMA vaults are likely, ultimately, to experience high hydraulic gradients and so it is suggested that the lowest limiting crack width of 0.05mm would be more appropriate.

4.6.4. Minimum Reinforcement Requirements

The use of FRP reinforcement with higher tensile strength than conventional steel has the effect of reducing the amount of reinforcement required to control cracking, termed $A_{s,min}$.

The minimum area of reinforcement is calculated using the expression:

$$A_{s,\,min} = \frac{k_R k_c k\, \alpha_{ct} f_{ctm}(t)}{f_{yk}} A_{ct} \tag{4}$$

where: $A_{s,min}$ is the minimum area of reinforcing steel within the tensile zone (mm^2/m)
A_{ct} is the area of concrete within the tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

f_{yk} is yield strength of the steel

$f_{ct.m}(t)$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur, estimated according to the expressions in Cl 3.1.2 of EN1992-1-1. For assessment of early-age cracking it is common to use the 3-day value but where thermal analysis is undertaken, or relevant historical data indicate that cracking occurs at a different age, the tensile strength may be estimated accordingly. Similarly, for the assessment of cracking over the longer term, the 28-day value should be used unless thermal analysis or relevant historical data indicate otherwise.

α_{ct} is a coefficient which has been introduced to account for differences between the tensile test measured using a standard test specimen and the tensile strength in the structures.

If designing for a limiting crack width, the minimum reinforcement requirement is not necessary in this case, and crack control design is carried out assuming conventional steel reinforcement.

4.6.5. Crack Control Design

Calculations have been carried out based on comparisons of the thermal behaviour of the 2BMA concrete and the suggested ternary concrete mix (Table 12).

To enable a workable comparison, concrete placement temperatures were increased to 20°C, as the ternary mix would not crack at the assumed 10°C placement temperature discussed in Section 4.3.

The analysis assumes the following:

- Estimation of peak temperature (at 20°C placement temperature)
- Estimation of crack inducing strain in the 680mm thick walls at a restraint level of 0.42
- Estimation of minimum reinforcement required
- Crack control reinforcement required to provide a 0.05mm crack width at early age.

Table 12: Summary of crack width control comparisons.

Concrete Type	**2BMA mix**	**Ternary mix (55% GGBS)**
Assumed compressive strength (at 28 days)	C32/40	C 32/40
Thermal coefficient of expansion (mm/m/°C)	10	10
Peak temperature (°C)	57.5	50.9
Temperature drop (T1)	37.5	30.9
Crack inducing strain (microstrains)	75.5	57.7
Minimum reinforcement requirement (mm^2/m)	642	642
Crack spacing (mm)	599	830
Crack width	0.05	0.05
Reinforcement density for limiting crack width (mm^2/m)	2011	785
Reinforcement needed for limiting crack width (per face)	16mm bars at 100mm centres	10mm bars at 100mm centres

The above comparison shows that for the 2BMA mix used in the trials, 16mm bars at 100mm centres would be required to control crack width to 0.05mm, compared with 10mm bars at 100mm centres for the assumed GGBS mix. This represents a reduction in reinforcement of 61% for the suggested ternary mix.

In conclusion, where reinforcement is considered necessary to control early thermal cracking, the ternary concrete mix containing moderate to high replacement of GGBS, will assist in efficiently controlling cracking.

5. Concrete Evolution

5.1. Treatment of Concrete Evolution in SFR-PSU

SKB's treatment of concrete evolution and degradation in the concrete caissons of the 2BMA vault in the SR-PSU safety assessment (SKB, 2015) is based on modelling by Höglund (2014).

In the assessment, concrete degradation is taken into account by assigning different transport properties (hydraulic conductivities, effective diffusivities, porosities, K_d) to different time intervals in accordance with the predicted concrete degradation (Table 13, Table 14 and Table 15). This approach is reasonable.

Table 13: Progression of hydrological conditions in the near field of the 2BMA vault over time in the Global Warming Case of the Main Scenario of the SR-PSU assessment, and the Accelerated Concrete Degradation Case of the Accelerated Concrete Degradation Scenario. Black = No flow; Blue = Moderately degraded concrete, $K=10^{-7}$ m/s; Yellow = severely degraded concrete, $K=10^{-5}$ m/s; and Orange = completely degraded concrete $K=10^{-3}$ m/s. "F" denotes fracture flow (Global Warming Case from Table 4-1 in SKB, 2014; Accelerated Concrete Degradation Case from Table 4-7 in SKB, 2014).

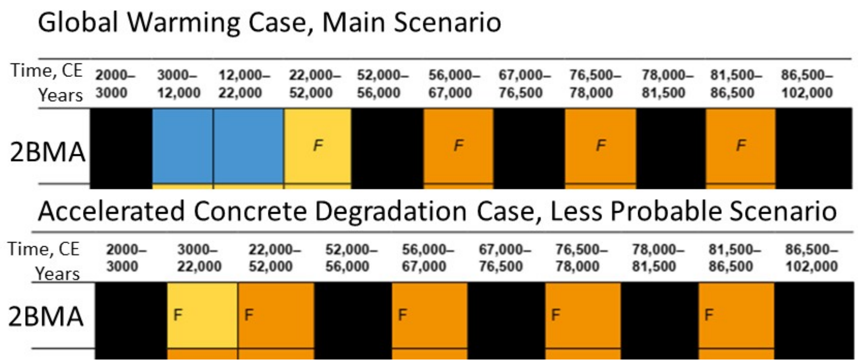

Table 14: Effective diffusivities and porosities for different time periods used for construction concrete in the 2BMA vault in the Global Warming Case of the Main Scenario of the SR-PSU safety assessment (from Table 4-2 and Table 4-3 in SKB, 2014) and the Accelerated Concrete Degradation Case of the Accelerated Concrete Degradation Scenario of this safety assessment (from Table 4-8 and Table 4-9 in SKB, 2014).

Time (Years CE)	Global Warming Case		Accelerated Concrete Degradation Case	
	De (m^2/s)	Porosity (-)	De (m^2/s)	Porosity (-)
2000 - 2100	3.50E-12	0.11	3.50E-12	0.11
2100 - 2500	5.00E-12	0.14	5.00E-12	0.14
2500 - 3000	5.00E-12	0.14	5.00E-12	0.14
3000 - 12000	5.00E-12	0.14	5.00E-12	0.14
12,000 - 22,000	5.00E-12	0.14	1.00E-11	0.18
22,000 - 52,000	1.00E-11	0.21	2.00E-10	0.50
52,000 - 100,000	2.00E-10	0.25	2.00E-10	0.50

Table 15: Concrete degradation states in the 2BMA vault for different time periods, used as a basis for selecting different Kd values to model sorption in the Global Warming Case of the Main Scenario of the SR-PSU safety assessment (from Table 4-4 of SKB, 2014).

Time CE (Years)	Concrete Degradation State	pH and Key Process
2000–4000	I	>12.5, leaching of K- and Na- hydroxides
4000–7000	I	
7000–22,000	II	12, dissolution of portlandite ($Ca(OH)_2$
22,000–34,000	II	
34,000–58,000	II	
58,000–102,000	II	

The possibility that the concrete might degrade more rapidly is considered in the Accelerated Concrete Degradation Scenario, which SKB consider to be a less probable scenario. The reviewers concur with SKB's judgement in this regard. In this scenario, hydraulic conductivities, diffusivity and porosity of the concrete increases earlier or to a greater extent than in the Global Warming calculation case of the Main Scenario (Table 13, Table 14 and Table 15). SKB (2014, 2015) is unclear how sorption was treated in the Accelerated Concrete Degradation Scenario, but it appears to have been the same as in the Global Warming calculation case of the Main Scenario.

It can be seen from the above tables that in the Global Warming calculation case of the Main Scenario, the flow-limiting function of the concrete caissons is maintained for at least 20,000 years. The justification for this provided by SKB is that transformations of cement phases within the concrete structure maintain its low permeability (Höglund, 2014).

The Accelerated Concrete Degradation Scenario results in calculated annual doses that are up to 50% greater than those calculated for the Main Scenario. However, the conditional doses (without assignment of scenario probability) do not exceed the risk criterion, although some approach it to within a factor of two.

5.2. Concrete Degradation in SE-SFL

SKB has undertaken work on concrete degradation in support of its development of proposals for a repository of long-lived low- and intermediate- level waste, the SFL (Idiart and Shafei, 2019; Idiart et al., 2019a,b). A review of this work for SSM has been carried out previously by Quintessa (Metcalfe et al., 2021). While strictly outside the scope of the present review, SKB has based its initial evaluation of the SFL on its work for the SFR. There are many design similarities between the proposed SFL and the SFR extension. Therefore, it is relevant to consider SKB's work on the SFL here.

Whereas SKB does not appear to have considered degradation of the concrete mix used in the caisson tests for the 2BMA vault of the SFR extension (Mårtensson and Vogt 2019, 2020), the development work conducted by SKB for the SFL has considered several alternative concrete mixes:

- Anläggningscement (i.e. CEM I 42.5 N - SR 3 MH/LA) with a w/c ratio of 0.47;
- Anläggningscement with a w/c ratio of 0.63 and 320 kg/m3 concrete;
- Anläggningscement with a w/c ratio of 0.63 and 280 kg/m3 concrete;
- Bascement Slite (i.e. CEM II/A-V 52.5 N) with a w/c ratio of 0.47 and 12.3 wt% fly ash; and
- Anläggningscement containing limestone and dolomite addition with w/c of 0.49.

Based on modelling, Idiart et al. (2019b) concluded that:

> *"The case which considers the addition of limestone filler is the concrete mix that has the best performance. According to the model results, this improvement is more related with the initially lower porosity compared to the rest of cases than to the chemical composition of the cementitious system."*

A question therefore arises as to whether SKB has taken into account the results of work on concrete degradation for the SFL in its work to develop the design for 2BMA. From the reviewed documentation it is not clear that this has been done.

5.3. Modelling of Concrete Degradation

Concrete degradation was modelled extensively by Höglund (2014). The modelling is thorough, but the concrete composition used differs from that now proposed for the caissons (Table 1), which was tested as described in Mårtensson and Vogt (2019, 2020). Unfortunately, the concrete composition given in Höglund (2014) does not allow de-tailed comparison with that in Table 1. However, it is clear that there are considerable differences (Table 16). Notably the composition considered in Höglund (2014) does not contain fine limestone. Reflecting this difference, potentially there could be differences in the degradation pathways modelled by Höglund (2014) and the actual degradation pathways were the caissons to be constructed using the concrete mix of Lagerblad et al. (2017).

Notably, assuming that there is sufficient SO_4 present in the porewater, the inclusion of limestone could result in a greater risk that formation of thaumasite ($Ca_6Si_2(OH)_{12}(CO_3)_2(SO_4)2.24\ H_2O$) could occur, leading to enhanced loss of C-S-H and consequently cohesion. This possibility was recognised by Höglund (2014) when reviewing previous work on cement evolution. Whether any differences in degradation between the concrete composition considered by Höglund (2014) and the composition recommended by Lagerblad et al. (2017) would be significant for the performance of the caissons in the 2BMA vault is uncertain, but SKB needs to explore this issue.

Table 16: Comparison between the concrete composition recommended by Lagerblad et al. (2017), and used in the caisson tests described in Mårtensson and Vogt (2019, 2020), and the concrete composition considered by Höglund (2014) in the modelling of concrete degradation.

	Lagerblad et al. (2017)		Höglund (2014)	
Constituent	**Description**	**Proportions (kg/m^3)**	**Description**	**Proportions (kg/m^3)**
Cement	Degerhamn Anläggningscement/ Cementa AB	320	Degerhamn Anläggningscement/ Cementa AB	350
Filler 1 (2µm)	Omyacarb 2GU/ Omya	130	None	
Filler 1 (10µm)	Myanit 10 / Omya	33.3	None	
Water		156.8		164.5
Coarse aggregate	Crushed rock (16-22 mm)	393.3	Crushed rock (16-32 mm)	535
Coarse aggregate	Crushed rock (8-16mm)	425.7	Crushed rock (8-16mm)	374
Coarse aggregate	Crushed rock (4-8mm)	92.0	Crushed rock (0-8mm)	920
Fine aggregate	Crushed rock (0-4mm)	840.9	-	
Superplasticiser	Master Glenium Sky 558 / BASF	1.30	Sika Plastiment BV-40	0.5%
Superplasticiser	Master Sure 910 / BASF	1.70	-	
Retarder	Master Set RT 401 / BASF	0.96	Sika Retarder	0.05% - 0.2%

The physical and chemical evolution of the concrete is dominated by:

- progressive leaching of cement minerals (initially of Na- and K-hydroxides, then of portlandite, then of CSH phases (but see below));
- expansive formation of secondary cement minerals (ettringite and thaumasite); and
- fracturing (partly driven by weakening due to leaching, and stresses caused by formation of high-volume phases.

Example output from the coupled models of concrete evolution in Höglund (2014) is shown in Figure 45. It is noteworthy that this figure shows that portlandite is leached after c. 15,000 years, the SR-PSU assessment considered that portlandite would remain for the entire 100,000 years of the assessment (Table 15).

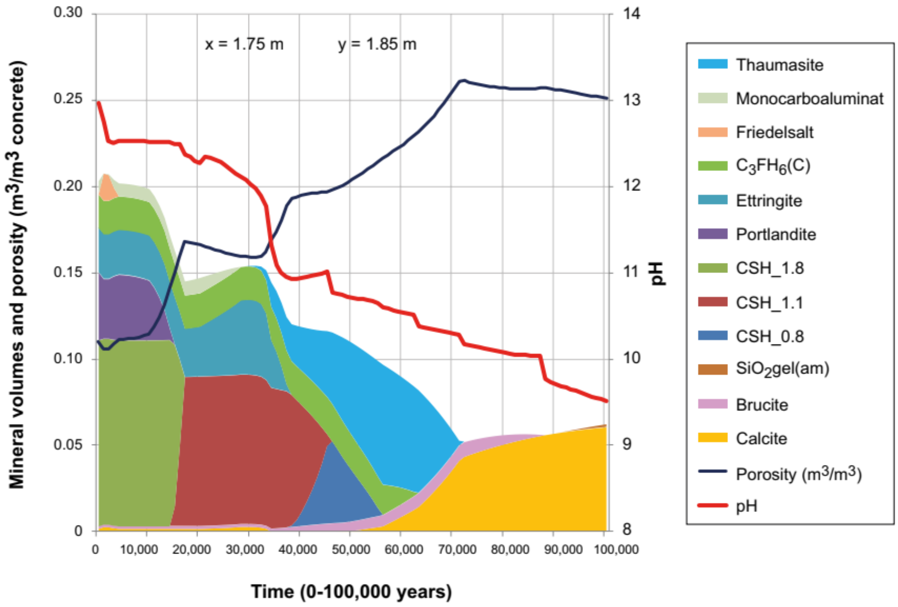

Figure 45: Modelled evolution of mineral volumes, porosity and pH in concrete over the full 100,000 years safety assessment period (linear scale) at the centreline of the concrete lid and the left-hand side concrete wall in the 2BMA vault (from Höglund, 2014).

The evaluation of concrete degradation by Höglund (2014) also included detailed analysis of fracture formation and its coupling to concrete alteration. This work concluded that the hydraulic properties of a fracture will not change until the portlandite-depleted zone has penetrated the whole thickness of the concrete barrier. It is stated that for 2BMA, where the fractures in the concrete barrier have apertures less than 0.1 mm, the water flow rate in the fractures is sufficiently low to ensure that this will not occur during the first 20,000 years after closure.

The numerical modelling of Höglund (2014) used the water compositions in Table 17.

Table 17: Water compositions used in the analyses of Höglund (2014).

Parameter (mg/L)	Salt-Water Period			Fresh-Water Period		
	Proposed	Min	Max	Proposed	Min	Max
HCO_3^- (Alkalinity)	100	40	110	300	170	540
SO_4^{2-}	500	20	600	50	3	110
Cl^-	5000	3000	6000	45	5	1000
Na^+	2500	1000	2600	100	20	200
K^+	20	6	30	4	0.2	10
Ca^{2+}	430	200	1600	35	25	140
Mg^{2+}	270	100	300	9	3	10
pH	7.3	6.5	7.8	7.49	6.7	8.7
Eh (mV)	Reducing	-400	-100	Reducing	-400	-100
Si (as SiO_2(aq))	5.66	-	-	5.9	-	-
Electrical balance (%)	-0.04	-	-	-0.08	-	-

These water compositions can be considered to bracket the range of groundwater compositions presently at the depth of the SFR and above. However, more saline water exists at much greater depth (Ca-Na-Cl dominated with c. 50,000 mg/L Cl, below ~700 metres, based on experience from the Forsmark site; SKB, 2013). There appears to have been no consideration of the effect such water would have on concrete degradation were this water to upcone to the level of the SFR in future. While the reviewers consider this possibility of upconing to be unlikely, it would be helpful for SKB to comment on it explicitly and to reference evidence that supports this process being unimportant.

The approach to the more recent modelling of concrete degradation for the SFL reported in Idiart et al. (2019b) was very similar to the approach used by Höglund (2014). The modelling approaches of the two studies are compared in Table 18.

Table 18: Comparison of cement degradation models in Höglund (2014) and Idiart et al. (2019b).

Model Characteristic	Höglund (2014)	Idiart et al. (2019b)
Groundwater flow	HSTD3	COMSOL*
Geochemical reactions	PHREEQC (invoked as a sub-routine of PHAST)	PHREEQC*
Activity correction	Extended Debye-Hückel	Extended Debye-Hückel
Database	Cemdata07	Cemdata07
Dimensionality	2-D	1-D and 2-D
Reaction kinetics	No	No

*Coupled via a software interface iCP

The approaches are reasonable, but:

- The models use operator splitting in which flow and chemistry are solved sequentially at each time step (they are not strictly fully coupled).
- The models do not take into account reaction kinetics.
- The thermodynamic models are valid for the range of water salinities considered, but would not be valid for brines as these are too saline to justify use of the Debye-Hückel approach for activity corrections. While it seems unlikely that there would be upconing of brines to the level of the SFR, it would be helpful for SKB to state this explicitly, together with references to supporting evidence.
- More recent thermodynamic data for cement are available; CEMDATA7 was the database used in the modelling by both Höglund (2014) and Idiart et al. (2019b), whereas the latest version of CEMDATA is CEMDATA18.1.
- SKB's modelling does not take any account of the effect of the repository on groundwater.

Comparative modelling in Metcalfe et al. (2021) showed full coupling and inclusion of kinetics does not result in large changes to modelled cement evolution.

6. Conclusions

6.1. Reviewed Documents

An in-depth review of the work carried out by SKB on the 2BMA disposal vault design has been carried out. The following main reports published by SKB have been reviewed.

- TR-18-12: Caissons for 2BMA: Large scale tests of design and material. (Mårtensson, and Vogt, 2019).
- TR-20-09: Concrete Caissons for 2BMA: Large scale test of design, material and construction method. (Mårtensson, and Vogt, 2019).
- R-17-21: Development of structural concrete to the caissons of 2BMA (Lagerblad et al., 2017) (translated).
- Further developed design of the repository space 2BMA in expanded part of SFR. Report 1569813. 2017 (translated).
- R-13-40: The impact of concrete degradation on the BMA barrier functions (Höglund, 2014).

The following sub-sections give the conclusions from the review.

6.2. Mix Design

The main reports reviewed conclude that the selected concrete mix design (see Table 1) can be used on a production scale to cast large concrete structures. However, it is questioned herein whether the concrete mix is in fact optimised for the purpose of caisson construction. Concerns arise from the implementation of the selected concrete mix based on the reported observations during the trials.

Both TR-18-12 and TR-20-09 mention that the mix does not retain its fresh properties readily, and that rejection of batches, and significant adjustments to the batched concrete were required to ensure the concrete maintained its workability (i.e. consistence) during transport and placement. The principal reason for this is reported to be the use of limestone powder within the mix. Consequent problems with water demand, cohesion and retention of workability follow from this, as well as a certain degree of impracticalities in batching the mix.

Review of the mix concludes that despite achieving acceptable hardened mechanical properties, the selected concrete mix contains very high amounts of limestone fines, making the mix susceptible to problems in the fresh state, with significant reduction in workability, and risking consistent quality during construction.

There are alternative mixes with lower proportions of limestone fines that are considered likely to produce better results in relation to fresh property retention and thermal control than those obtained by SKB using the mix given in Table 1, and consequently increased reliability and reduced construction risk. The question arises as to why SKB has not reported consideration of such mixes – or if SKB has considered them, what was the justification for rejecting them. Using a mix with ground granulated blast furnace slag (GGBS) as a cementitious replacement material (termed a “ternary” blend concrete) would probably avoid the problems encountered by SKB. Did SKB consider such a mix? If not, what was the reason? Furthermore, the inclusion of GGBS would potentially provide a range of additional benefits. Comparable test data and discussion of properties are provided in Section 4.2.5. A preliminary mix design for consideration of initial trials is presented in Table 4.2. Mixes of this type are being used in major civils works in the UK and have a favourable track record.

Batching of the concrete also requires careful re-evaluation. SKB has not provided details of how it would ensure consistent batching of the concrete during construction of 2BMA. An established fixed batching plant close to the site is considered desirable. High quality mobile wet mix batching plant solutions, using computerised weigh batching, effective moisture correction and forced action mixing of concrete would probably achieve the required results. Has SKB considered such an approach?

6.3. Cracking Risk

Although no cracks have been observed in any of the trials, crack risk in walls is considered to be a possibility in the analysis carried out herein (Section 4.3.6). Cracking would be uncontrolled without the use of reinforcement if it were to occur.

Comparisons in thermal behaviour between the selected 2BMA mix containing high limestone fines, and a ternary blend concrete shows that crack risk is reduced when the latter mix is considered, due to the lower heat generated during hydration.

Section 4.3 provides further detail and discussion.

6.4. Joint Design

The first trials constructed both a slab and wall integrally (in one single cast), and a separate slab and wall cast. The integral casting of the walls was reported to be problematic, resulting in poor surface finish of the base slab. This method is currently considered a risk to consistent quality of caisson construction.

It was concluded in the first report (TR-18-12) that the hydraulic conductivity of the joints and the interface between the joint seal and the concrete was found to be closely similar to that of the bulk concrete. SKB therefore concluded that separate casting of slabs and walls would be suitable. However, the present review has not found sufficient evidence to support SKB’s conclusion in the reviewed reports. Although separate casting of base slab and walls has the potential to reduce construction risk compared with integral casting, further consideration is recommended for the design for joint waterproofing.

In the case that the walls are cast separate to the slab, the reviewed reports state that careful preparation of the joint surface prior to casting would improve the hydraulic properties of the joint. The reviewer agrees with this assertion but highlights the dependency on workers implementing construction procedures to a consistently high standard to achieve high quality concrete at the joint. In the second report (TR-20-09), localised issues with poor joint preparation are mentioned. In addition, no compelling evidence is presented to show that the concrete at the joint is similar to the bulk concrete. Joint preparation alone is not considered to provided sufficient joint waterproofing quality.

It is concluded that a more reliable procedure would be to cast the slab then the walls and control the other aspects of the construction risk. This control would include both high quality joint preparation (and associated Quality Assurance procedures), along with a well-designed waterproofing barrier system.

Design of water resisting structures requires at least two levels of defence and in the case of the caissons to be constructed in the 2BMA vaults maybe even three levels of defence would be desirable:

- good joint preparation;
- waterproofing barrier design; and
- crack control.

From the information presented in the reviewed reports it is not clear that SKB has plans to implement such "defence in depth".

Some examples of waterproofing barriers are provided in Section 4.4.3. Crack width control is discussed in Section 4.6.

6.5. Tie Hole Review

The pressure exerted by the fresh concrete during placement requires structural strengthening of formwork systems to allow them to remain in dimensional stability. Formwork ties are used to hold formwork together during the process of concreting most vertical units.

Systems alternative to tie rod formwork would be the single sided / propped formwork system, whereby formwork pressures are resisted by the support frame which is bolted to a sturdy foundation. The alternative would be to brace the formwork between walls (either as constructed or within cavern walls).

In addition, tie-less concrete construction can be carried out using a slipform system. Slipform concrete processes is a specialist method of construction of vertical concrete structures, or the production of horizontal paving structures (e.g. slabs, barriers etc.), and is one of the fastest methods of concrete construction. The concrete is effectively extruded from the moving formwork.

The disadvantages of these systems are potentially the space requirements within the vaults, along with bespoke design and construction cost requirements.

Tie rod systems for wall construction appear to be a more preferable option in terms of constructability.

There are various types of tie rod that range from lightweight to heavy duty, and those that can be removed and reused. Those used for more heavyweight purposes are systems such as threaded tie, taper tie and she-bolt tie systems. These are explored in Section 4.5.3.

In conclusion, there are alternatives to the tie-rod systems used by SKB in the trials that would appear to be more advantageous. It is not clear whether SKB considered these systems. If so, SKB should justify why the tie-rod systems used in the trials were selected.

Filling of the holes also allows some flexibility. The second stage trials report attempted the use of a rudimentary pumped system for tie hole filling, with various degrees of success. Alternatives that could potentially be more suitable are:

- Expanding polyurethane or acrylic grouts.
- Post hardening expansive cementitious grouts (e.g. calcium sulfo-aluminate grouts).
- Pressure injected cementitious grouting procedures.

6.6. Reinforcement

SKB state that no steel reinforcement should be used in the design of the caissons and accordingly these have been designed without structural reinforcement of any kind. However, SKB has not demonstrated that early thermal cracking and drying shrinkage would not be problematic in the absence of reinforcement. Depending on the concrete used, this cracking and drying may require controlling by reinforcement. Several options are available and are reviewed briefly in this report. Has SKB considered these (or other options)? If so, what was the justification for not using them / proposing them given the potential benefits?

Most material types for non-ferrous reinforcement are based on the approach using particular fibre types that are encased in a resinous matrix. Common to most of these materials is the fact that the reinforcing typically has higher tensile strength but lower stiffness when compared with more conventional carbon steels. The mechanical behaviour of FRP reinforcement differ to that of conventional steel reinforcement. FRP materials are anisotropic in nature, characterised by high tensile strength in only one direction but with low ductility. They are elastic until failure. Their use as crack control reinforcement should therefore be generally well suited.

Typical fibre types are:

- Fibre reinforced plastic (FRP/GFRP)
- Aramid fibre composite (AFRP)
- Carbon fibre composite reinforcement (CFRP)
- Basalt fibre reinforcement (BFRP)

Mechanical properties of the above materials are presented in Table 11.

Guidance on limiting crack widths for water-resisting structures are given in Bamforth (2018) (CIRIA C766). For structures with potentially high hydraulic gradient, crack width limits of 0.05mm would be appropriate. Assuming the cracks are not active (i.e. minimal dynamic movements), it is likely that the cracks will self-heal.

Both the selected 2BMA mix and the example alternative ternary mix presented in this report have the potential to crack under the right conditions (e.g. higher temperatures). This cracking would be uncontrolled without the use of reinforcement.

It is therefore considered that the use of non-ferrous reinforcement would benefit in limiting the crack width in the caissons. Calculations to determine the amount of reinforcement required were carried out for each concrete type, placed at conservatively higher temperatures than during trials. The calculations show that the ternary blend containing GGBS and smaller proportions of limestone fines would utilise around 60% less reinforcement than the selected 2BMA mix to control the cracking to 0.05mm.

6.7. Concrete Degradation

SKB has undertaken extensive studies of concrete degradation, as reported in Höglund (2014). These studies are based on literature reviews and numerical simulations and underpinned the SR-PSU safety assessment. However, the work in Höglund (2014) is rather old and considered a different concrete composition to the one now proposed by SKB for use in 2BMA caisson construction, as described in Lagerblad et al. (2017) and Mårtensson and Vogt (2019, 2020). Notably, the concrete mix now proposed contains a significant proportion of limestone fines. It appears that SKB has not conducted a de-tailed investigation of the long-term evolution of this concrete. Consideration needs to be given to the possibility that the concrete now being considered by SKB for use in caisson construction may degrade differently to the concrete considered by Höglund (2014) and the SR-PSU assessment (SKB, 2015).

More generally, if a different composition to that considered in Höglund (2014) or Lagerblad et al. (2017) is used, SKB will need to show that its future evolution and possible degradation is well understood and that it meets the performance requirements from a radiological safety perspective.

While not strictly in the scope of this review task, SKB's modelling in support of its proposed repository for long-lived low- and intermediate- level waste was reviewed briefly, as this proposed repository has many similarities with the SFR. In support of its work to develop the SFL, SKB has undertaken studies of concrete degradation that are more recent than those it has carried out for the SFR (Idiart and Shafei, 2019; Idiart et al., 2019a,b). These studies have considered several alternative concrete mixes, including a mix containing limestone that appears to be similar to the one used in the caisson tests aimed at the 2BMA vault. This raises the question as to whether SKB has taken into account work undertaken for the SFL in its development of the 2BMA vault? If so, how has this been done? Where is it reported?

6.8. Overall Assessment of SKB's Development of Concrete for Caisson Construction

6.8.1. Quality of Documentation

SKB has undertaken an impressive range of work, which is generally clearly documented. All the reviewed reports are logically structured. However, relationships between the documents are less clear. In particular the way in which the concrete composition and caisson design in Mårtensson and Vogt (2019, 2020) were developed from the concrete composition and design considered in SR-PSU (SKB, 2015) could be more clearly presented.

6.8.2. Progress Made Relative to SSM's Expectations

2BMA Feasibility of Construction

In the reviewer's opinion, SKB has not demonstrated confidence that the caissons can be constructed using the proposed methods and materials in a manner that will achieve the required performance. While arguably the problems identified in trials undertaken at Äspö could be addressed by modifications to the methods employed, it seems unlikely that this could be done using presently proposed concrete mix. Whatever concrete mix is eventually used, additional work is needed to ensure that the required quality can be achieved and maintained at the scale of the 2BMA vault.

Long-Term Performance of Concrete Barriers

SKB appears not to have taken into account the effect of chemical processes on the mechanical strength of the concrete; analyses in Höglund (2014) investigate the reverse relationship. According to Babaahmadi (2015) and Babaahmadi et al. (2015) the strength properties of concrete where the portlandite has been completely leached is independent of the initial water / cement ratio, although complete leaching of portlandite results up to a 70% decrease in strength. The rate of leaching of the portlandite will be coupled to the fluxes of water through any fractures that develop, as well being dependent on the concrete composition and the groundwater composition.

Höglund (2014) modelled concrete degradation using both saline groundwater compositions (Cl up to 6000 mg/L) and fresh / brackish groundwater compositions (Cl up to 1000 mg/L). These compositions bracket much of the water chemical variability presently around the SFR site. However, the possibility that more saline deeper groundwater may upcone into the SFR in future appears not to have been considered. It is recognised by the reviewers that this upconing is unlikely, but it would nevertheless be helpful for SKB to state that this is the case and to provide supporting evidence. Notwithstanding this, a bigger issue at present is that SKB appears not to have investigated degradation of concrete of the composition proposed in Lagerblad et al. (2017), as used in Mårtensson and Vogt (2019, 2020). However, SKB has undertaken more recent, extensive work on concrete degradation for the SFL (Idiart and Shafei, 2019; Idiart et al. 2019a,b).

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