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MYCONCRETE

THE BULLETIN OF THE AMERICAN CONCRETE INSTITUTE - MALAYSIA CHAPTER
(E-bulletin)

Highlight!

Upcoming Event

ACI-Malaysia Chapter 26th General Meeting –
14th April 2023

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MyConcrete:
The Bulletin of the American Concrete Institute – Malaysia Chapter

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AMERICAN CONCRETE INSTITUTE-MALAYSIA CHAPTER

26TH ANNUAL GENERAL MEETING

14th April 2023, Friday

5.00 PM - AGM
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ARTICLE

Challenges in Corrosion Management for Marine Infrastructure





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Abstract: This article discusses the challenges faced in managing corrosion in marine infrastructure, which is often made up of concrete parts, including docks, bridges, and maritime bases. Concrete damage caused by corrosion from steel reinforcement is a common occurrence in marine infrastructures, and maintaining the quality and original form of reinforced concrete in marine environments is a difficult challenge due to the complex mechanisms of concrete deterioration. Chloride-induced corrosion of reinforced concrete is the most common cause of corrosion in reinforced concrete structures exposed to chloride in the marine environment. Researchers have previously explored ways to restore and retrofit reinforced concrete in marine applications to prolong its service life, with the highlight parameters being the materials used, concrete cover thickness, compressive strength, and chloride migration coefficient. Increasing the compressive strength of the concrete appears to have the greatest impact on improving reinforced concrete in marine applications.

Marine infrastructure generally involves concrete parts as general infrastructure such as in docks, bridges, maritime bases, terminals, platforms, submarine pipelines and etc. [1] – [6]. Concrete damage caused by corrosion from steel reinforcement is a common occurrence in marine infrastructures [1], [3], [5], [7] – [9]. A common durability issue faced by reinforced concrete structure is to maintain its physical and mechanical properties while being exposed to marine environments that cause the deterioration of reinforced concrete structures [6], [10] – [12]. The main mechanisms of concrete deterioration are chemical, physical, and mechanical in nature, which are all intricately linked.

Since concrete structures are often exposed to a variety of conditions, maintaining the quality, original form and serviceability of reinforced concrete in the marine environment present more difficult challenges [4], [13], [14]. The primary mechanisms of the concrete deterioration can occur through concrete surface (abrasion, erosion and cavitations), reinforcement (corrosion of steel), aggregate (alkali-aggregate reactions, freezing and thawing) and cement-based matrix (seawater attack, salt crystallization, carbonation, acid attack, sulphate attack) as depicted in Figure 1 [6], [14] – [16].

Figure 1: Example cases of primary mechanism of the deterioration by (a) cavitations, (b) sulphate attack, (c) reinforcement, (d) carbonation and (e) freezing and thawing

Carbonation, chloride, and stray or electric current are just a few of the factors that contribute to reinforced concrete deterioration [4], [5], [6], [9]. Diffusion of chloride into reinforced concrete is one of the most significant mechanisms of degradation, contributing to corrosion of the embedded reinforcing steel [17] – [19]. Chloride ingress through capillary action via airborne salt spray and/or wetting and drying with seawater is the primary cause of chloride corrosion in marine concrete structures [20], [21]. Chloride concentrations of 0.2 to 0.4 percent or higher that is transported into the concrete via sorption and ionic diffusion can cause corrosion [5], [20], [22].

Chloride-induced corrosion of reinforced concrete is the most common cause of corrosion in reinforced concrete structures exposed to chloride in the marine environment. The initiation and propagation of reinforcement corrosion, which lead to cracking, spalling, and the loss of load bearing capacity of reinforced concrete structures, had been reported to be a serious durability concern in reinforced concrete structures across the world. In order to protect the reinforcing steel from corrosion, solutions for concrete repair and retrofitting in marine applications should be understood.

Chloride ions enter the concrete by diffusing through the pores. When chloride ions come into contact with steel reinforcement concrete, they can cause corrosion, which accelerates the oxidation process. The passive film over wide areas of the reinforcement can be destroyed by very high levels of chloride where corrosion areas are usually called pitting corrosion and the area of corrosion by penetrating attacks surrounded by non-corroded areas is called pits. Chloride attack occurs when chloride ions are present in concrete in a marine environment or when de-icing salts are used. Since chloride ions are not consumed in the first reaction, Equations (1) and (2) describe a chemical reaction involving a continuous phase of chloride attack, while the chloride ions are free to react again in the second reaction (Hong et al., 2017; Zuquan et al., 2018). As a result, chloride ions have the potential to induce corrosion, which is one of the most harmful and damaging mechanisms in reinforced concrete.

$$\mathrm{Fe^{+++} + Fe^{++} + 6Cl^{-} \rightarrow FeCl_3 + FeCl_2} \qquad (1)$$

for hydrolysis

$$\mathrm{FeCl_3 + FeCl_2 \rightarrow 6Cl\text{-} + Fe(OH)_2 + Fe(OH)_3} \qquad (2)$$

Many researchers have previously looked at ways to restore and retrofit reinforced concrete in marine applications to prolong its service life [2] – [6], [15]. The effect of the materials used, concrete cover thickness, compressive strength, and chloride migration coefficient are the highlight parameters that have been considered by previous researchers with the aim of extending the service life of concrete in marine environment.

The impact of increasing the thickness of the concrete cover on the barrier to various aggressive agents moving towards the reinforcement and the time it takes for corrosion to start has been thoroughly studied in conjunction with other highlight parameters [14], [18], [21], [23]. The major influencing factors on minimum cover thickness have previously been reported to be pozzolanic material form, water/cement ratio and replacement level, and exposure conditions

[24]. However, the thickness of the cover must not exceed the limit (65 – 75 mm), as a thick layer of concrete cover has high risk of cracking and acts as a barrier to various aggressive agents moving towards the reinforcement and lengthening the time for corrosion to be initiated [5].

Based on the main parameters of this solution, the compressive strength appears to have the greatest impact on improving reinforced concrete in marine applications [23], [25], [26]. High compressive strength leads to an increased resistance against chloride ingress which extends the service life [22] – [24]. In marine application, high compressive strength could increase the durability and bond strength between concrete and steel reinforcement, reducing the time it takes for corrosion to be initiated as well as extending the service life of reinforced concrete [5], [21], [26]. Previous researchers have found that a low water/cement ratio, increased aging, and a decrease in penetration depth all reduce chloride migration coefficient, thereby reducing reinforced concrete damage or corrosion initiation [22], [24], [27].

Besides, the service life of reinforced concrete is strongly dependent on chloride migration coefficient, where the chloride migration coefficient should be the lowest. Several factors, including the water/cement ratio, type, quantity of cement used, chloride ion binding, materials, and environmental factors, all play a role in extending the service life of reinforced concrete structures subjected to chloride migration coefficient [23], [27] – [29].

In terms of the challenge and risk that the marine industry faces, geopolymer concrete can be a good solution for longer service life construction in a harsh chloride setting because its efficiency is equivalent to that of standard concrete. Geopolymer concrete can provide excellent mechanical properties as an alternative to Portland cement concrete with the appropriate mix design and formulation production [30], [31].

In 2018, Kim et al. [22] studied the performance of reinforced concrete for marine environment by using three different types of cement with supplementary cementitious materials such as fly ash and ground granulate blast slag. The chloride diffusion coefficients obtained were 1.39 x 10-12 m^2/s, 4.21 x 10-12 m^2/s, and 4.21 x 10-12 m^2/s for CEM I, CEM III/A (cement + slag) and CEM II/B-V (cement + fly ash) concrete, respectively (Figure 2). Although CEM I concrete had the highest resistance, the CEM III/A (cement + slag) and CEM II/ B-V (cement + fly ash) still have the potential to be used as binder in resisting chloride ingress which is good in slowing down the corrosion process. Plus, supplementary cementitious materials can resist chloride transport due to disconnected pore structures caused by continuous hydration and high chloride binding.

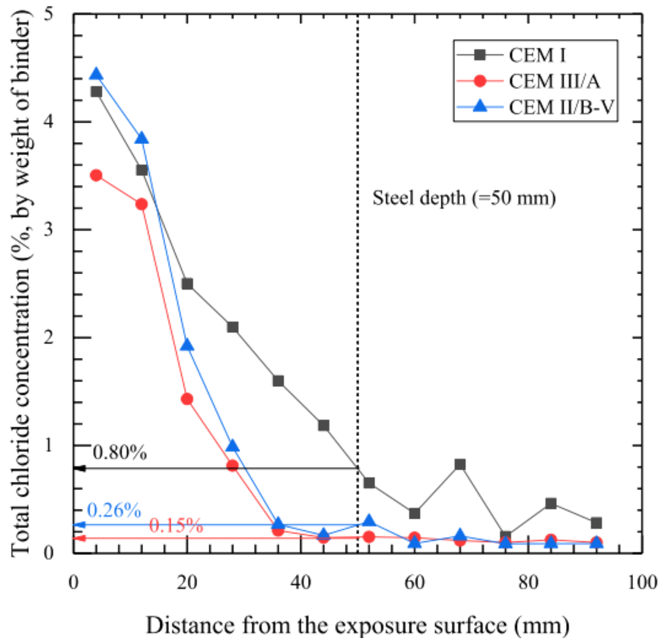

Figure 2: Chloride profiles through slabs after 18 years exposure [22]

In 2019, Darmawan et al., [32] studied the behaviour of a geopolymer concrete for beam application using high calcium content fly ash (FA) considered by curing the specimen in a sea splashing zone for 28 days in marine environment (Figure 3). Based on this finding, geopolymer concrete has demonstrated good compressive strength (24.78 – 35.09 MPa) with no effect from the sea in terms of beam crack pattern and development. However, further tests on chloride migration coefficient and cover thickness are required to understand the effect of the marine environment on the geopolymer concrete in terms of the important parameters.

Figure 3: Geopolymer concrete beam cured in seawater for 28 days [32]

In 2020, Noushini et al., [33] studied the effect of chloride diffusion of low-calcium fly ash-based geopolymer concrete prepared using different heat curing conditions for durability in marine environment. A reduction in migration coefficient was observed from the increase in heat curing temperature and duration. The chloride migration coefficients of the studied fly ash-based GPCs were significantly higher than that of OPC based concrete reported in the literature. Results showed that the chloride diffusion resistance and the chloride binding capacity of fly ash-based geopolymer concrete are very low (Figure 4). The values of non-steady-state migration coefficients for fly ash based geopolymer concrete ranged between 38 × 10^{-12} m^2/s and 79 × 10^{-12} m^2/s.

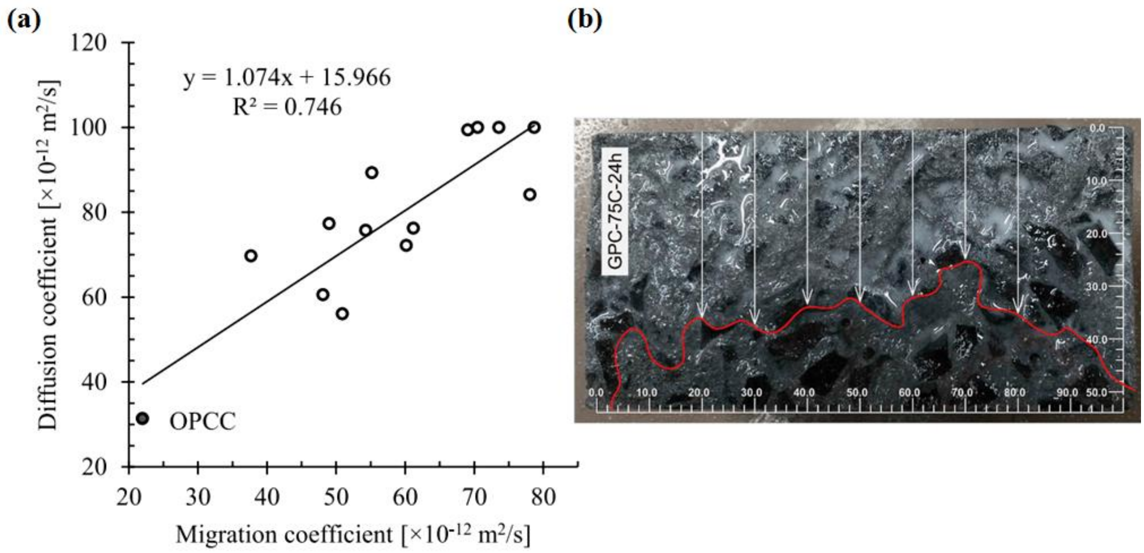

Figure 4: (a) Chloride diffusion coefficient versus chloride migration coefficient for GPCs and OPC (b) Example of chloride penetration depth test (NT BUILD 492) [33]

In conclusion, this article has reviewed the serious issue faced in marine infrastructure, specifically corrosion problem toward reinforced concrete. The review on various solutions to repairing and retrofitting the marine infrastructure which have been studied previously in terms of important parameters (materials used, concrete cover thickness, compressive strength, and chloride migration coefficient) has been conducted. Geopolymer concrete is thought to be one of the solutions, especially in improving corrosion management in marine application.

Geopolymer concrete has been proposed to be one of the selected materials for repairing and retrofitting solutions in marine application as geopolymer concrete has shown the capability to perform well in various circumstances. Nevertheless, geopolymer concrete has not yet been used fully in marine infrastructure as the solution for repairing and retrofitting of the reinforced concrete.

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Reprint from CI Magazine, Volume 39, No 12, Page 39-43

The Challenges of Achieving Compatibility in Concrete Repair

Volume changes, permeability, and electrochemical activity must be reviewed carefully

by Alexander M. Vaysburd, Benoît Bissonnette, and Kurt F. von Fay

During the last 25 years, our industry has gained considerable knowledge of the engineering aspects of concrete repair technology. Widely and generally used expert recommendations and specifications for concrete repair - “the law of the land” in many agencies and jurisdictions - now set forth what must be done to avoid trouble in repair performance. Nevertheless, we are still facing instances of gross deficiencies and an inability to meet the objectives of the designed service life for repaired structures. Much research and debate has been directed toward concrete repair durability. Considerably less attention has been devoted to compatibility/incompatibility aspects of repair durability. Even if most steps in the repair process are performed perfectly, the repair will likely fail prematurely if there are incompatibility issues.

The issue of compatibility in engineered repair composite systems is usually ignored or misinterpreted by design engineers. The main reasons include the lack of knowledge in this critical area, misleading guidance, and overall confusion among practicing engineers. Considering the significance of the problem and the handful of reported studies on durability and service life of repaired structures, it is astonishing that more questions than answers remain on the subject. Many concrete technologists, engineers, researchers, and other specialists simply may find compatibility too theoretical to study and address adequately.

A search for the word “compatibility” in textbooks and concrete repair publications ordinarily results in a blank. The available information on compatibility is very general, often containing misleading blanket statements and recommendations that rely on very simplistic design considerations. We all understand the negative results of incompatibility on repair performance, but knowing the results without clear understanding of the fundamental causes is no more satisfactory than postulation without verification. Seeking answers without the wherewithal to find them is equally frustrating.

The purpose of this article is in no way intended to criticize the “conventional wisdom” (we fully realize that a good musician is far superior to the best music critic). Rather, the article is our attempt to discuss some of the critical issues concerning the achievement of compatibility (deformational, permeability, and electrochemical) in a concrete repair composite system and its influence on repair durability, as well as fill important gaps in the industry’s basic knowledge of the subject.

We are quite aware that a significant body of opinion is likely to be at variance with the views set forth in this article. There are so many sources of uncertainty and disagreement that no one today can claim complete knowledge.

Compatibility in Concrete Repair

It has been long advocated that only repairing “like with like” can offer a durable solution and that for the repair material to be compatible with the existing concrete, it should have composition and properties like those of the substrate concrete. While this approach may appear to be sound, it carries many flaws. In many deteriorated structures, repairs are necessitated by inadequate quality of concrete, leading to problems such as extensive shrinkage cracking, alkali-silica reaction, and sulfate attack. To follow the “like with like” recommendations in a strict fashion, one would have to use similarly improper material for repair. Design engineers faced with such misleading interpretation of “deformational compatibility” may specify repair materials with properties as close as possible to those of the existing substrate concrete. This is not only illogical, but harmful. The temptation to seek parity of properties, to avoid property mismatches between repair material and substrate, is intuitively strong, but it contradicts the definition of compatibility in concrete repair.

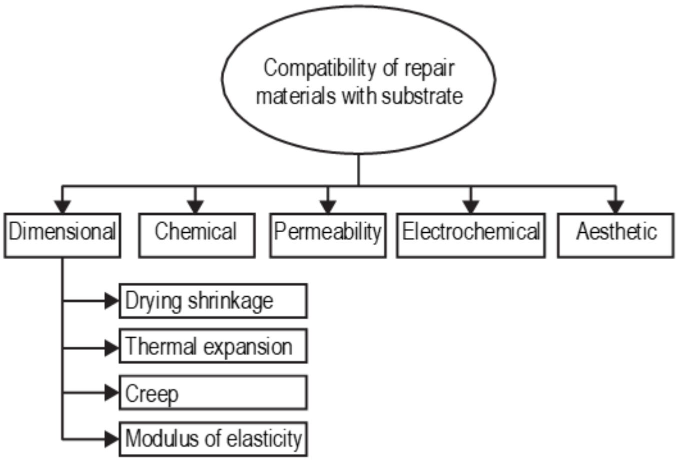

Fig. 1: Compatibility factors in concrete repair (adapted from Vaysburd and Emmons[1])

In some instances, it may be desirable for the repair material to have certain characteristics that are very different from the properties of the existing concrete. The concern should not be solely with the materials themselves, but also with the uses to which they are being assigned, with the inherently gray areas of overlap between the material per se, and the end product— the composite repaired structure.

Compatibility is generally defined as the capacity of two or more entities to remain together without undesirable after effects. Compatibility in repair systems can be defined as the balance of physical, chemical, and electrochemical properties, as well as volume changes between the repair and existing substrate. This balance ensures that the composite repair system withstands the stresses induced by loads, chemical and electrochemical effects, and restrained volume changes without distress and deterioration over a designed period of time.[1] Compatibility factors to be addressed in concrete repair are shown in Fig. 1.

Deformational compatibility

Hewlett[2] defined the notion of deformational compatibility in a repair composite system as a “stable interfacial coexistence.” Achieving deformational compatibility in a repair requires selection of appropriate materials such that the resulting composite exhibits long-lasting monolithic behavior.[3] Deformation mismatch may result in tensile stresses that cause either cracking of the repair material or debonding at the repair substrate interface (the transition zone). Cracking or debonding will negatively affect the durability and load-carrying capacity of the repaired structure. Potential sources of deformational mismatch include:

- Shrinkage (autogenous, drying) of the repair material relative to the substrate;
- Thermal deformation differences between repair and substrate materials;
- Differences in modulus of elasticity; and
- Differences in creep.

During service, incompatibilities in the form of dissimilar strength and moduli of elasticity between repair and substrate concrete can create problems, while drying shrinkage of repair materials may reduce long-term structural efficience by inducing high tensile strains in the repair. These strains can, at some point, trigger cracking that may reach the repair/substrate interface. Also, creep of the repair material under sustained stress may render the load-sharing capacity of the repaired member less effective with time. Mehta and Monteiro[4] introduced the concept of "extensibility of materials," which is directly related to deformational compatibility between repair materials and concrete substrates. The magnitude of shrinkage strains is a dominant factor, but it is not the only one that governs the cracking of the repair material. Other important factors include:

- **Restraint**—Restraint to volume changes limits changes in dimension, causing stresses in repair materials and possible cracking;
- **Modulus of elasticity—**The lower the modulus of elasticity, the lower the amount of the induced elastic tensile stress for a given magnitude of shrinkage;
- **Creep—**The higher the creep, the higher the amount of stress relaxation, and the lower the resulting tensile stress and risk for cracking; and
- **Tensile strength—**The higher the tensile strength, the lower the risk that the tensile stress will exceed the strength, reducing the likelihood of cracking of the material.

Cement-based materials are said to have a high degree of extensibility when they can be subjected to deformations without cracking. Unfortunately, the tensile strength of a cement-based material is low and cannot be increased substantially. Therefore, the only rational way to improve the extensibility, as defined by Mehta and Monteiro,[4] is by using a cement-based material with a low modulus of elasticity, high creep, and, perhaps most importantly, low shrinkage. Also, a successful cement-based material must develop tensile strength at a faster rate than shrinkage-induced tensile stresses. Otherwise, cracking occurs.

Most information on mechanical properties of concrete and other cement-based materials focuses on compressive properties. The importance of tensile properties is usually ignored. Hsu and Slate[5] provide a possible explanation for this oversight:
"...tensile strength of concrete is a quality different in nature from the compressive strength...Therefore, the tensile strength of bond, paste, mortar, and concrete deserves careful and thorough study by itself, separate from compressive strength."

Little consideration has been given to tensile creep in the design of new concrete structures, largely because of the difficulties related to the accurate measurement of these properties. In view of compatibility-based repair design, Vaysburd et al.[6] explained that it is not appropriate to use tensile properties of cement-based materials estimated from their compressive properties (such as elastic modulus and creep). While basing tensile properties on compressive properties is much easier than direct measurement, the practice can yield significant errors in evaluating the crack resistance of repair materials. Taking advantage of capacity of the repair material to deform in tension, especially its creep potential, could help prevent shrinkage-induced cracking and, thus, improve the durability of concrete repairs.

Extensive work by Bissonnette and Pigeon[7] and Bissonnette et al.[8] helped highlight the essential role of the tensile creep property of cement-based materials in repairs. The authors concluded that while shrinkage remains one of the major problems affecting the durability of thin concrete repairs, tensile creep can have quite the beneficial impact by relaxing the tensile stresses induced in the repair layer. Creep can prevent these stresses from overcoming the tensile strength of the material, thus avoiding cracking and, over the long haul, debonding.

The goal of the engineer must be to design the composite repair system in such a way that the inevitable property mismatches will not lead to failure before the service life objectives are met. It is important to emphasize that property mismatches between the substrate and the repair material are to some extent unavoidable. In addition, variability exists in the properties of the existing concrete and properties of the repair materials, as well as in the internal and exterior environments to which they are exposed. While the sources of variability occasionally cancel out, they generally have cumulative effects. In the quest for compatibility, the property mismatches and variability must be considered.

Permeability compatibility

Another widespread but mistaken philosophy in repair engineering has resulted in emphasis to use low permeability repair materials. While the use of low-permeability concrete in new construction is a key to achieving durability, the situation is more complex for repairs - there is little room for generalized rules for repair engineering. Instead, the philosophy should be “horses for courses.” Each situation is different and must be specifically studied and analyzed.

The use of low-permeability repair materials regardless of repair specifics can yield unsuitable choices, incompatibility problems, and eventual repair failures. In many situations, durability of the repair can be negatively affected when repair and substrate have different permeabilities. The material’s “micropermeability” (intrinsic permeability—between the joints and cracks) is important and must be as low as compatibility allows, but at least as much emphasis needs to be put on addressing the issues of macropermeability by specifying, for instance, an allowable shrinkage value. The transport of aggressive agents is controlled first by the macropermeability and then by the micropermeability. Cracking is one of the most critical factors in the overall permeability and durability of the repaired structures.

Aggressive agents take the route of least resistance, which, if present, is the network of cracks and microcracks. It is important to note that either a debonded repair or a repair with only a few “through” cracks will drastically offset the benefit of having a very low permeability repair material. Networks of microcracks connected with wider cracks originating from the repair surface play a greater role in promoting penetration of moisture and other external aggressive agents, thereby affecting durability, than the intrinsic permeability of the repair material. Cracking is one of the most critical factors in the overall permeability and durability of repaired structures (Fig. 2).

Engineering analysis and judgment is needed to define what degree of permeability of repair materials should be recommended for specific repair situations. Most likely, there is no single recommendation as to whether very low permeability or normal permeability compatible repair materials are more effective. It depends, in our view, on the particular transport mechanism in the repair system. Transport of substances through and in the repair systems is a very complex process, consisting of a combination of liquid flow through more or less connected macrocrack and microcrack systems, capillary transport, diffusion, and osmotic effects. The respective contribution of each process needs to be quantified in each situation. The effects of such variables as location of the repair in the structure, chemical environment in the composite repair system, amount and distribution of cracks in both materials (substrate and repair), temperature, moisture, and stresses need to be considered.

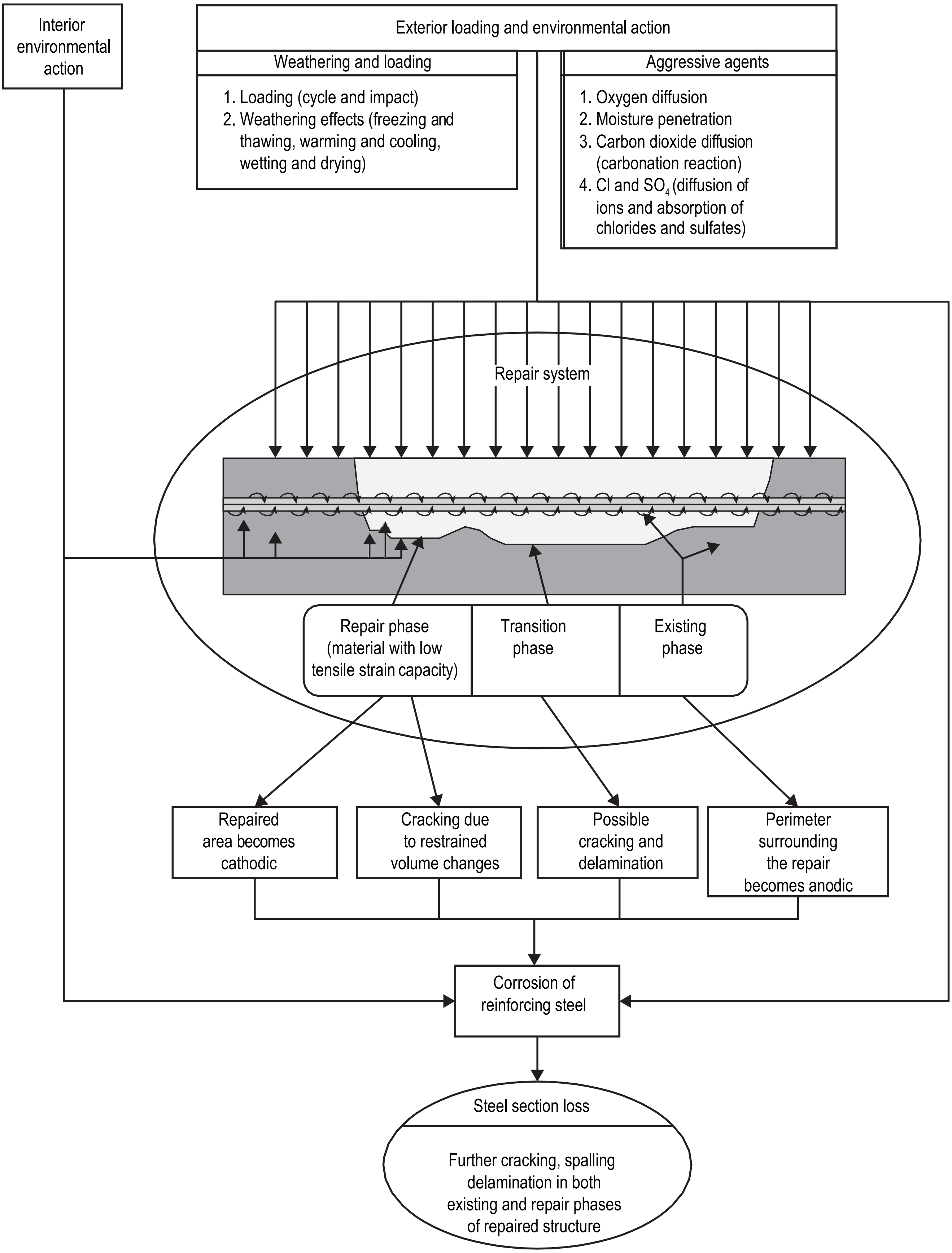

Fig. 2: Model of concrete repair failure caused by cracking[9]

Electrochemical compatibility

Electrochemical compatibility is often the most complex and critical issue for adequate performance of reinforced concrete repair. It is difficult to predict how a repair to a concrete structure will affect its electrochemical activity. Many parameters and interconnected processes are involved, such as the nature of the repair material, the nature and condition of the substrate, the differences in electrical potential, the exterior and internal (inside the repair system) environments and their interaction, and mass transport mechanisms. The risk of corrosion developing or even accelerating due to electrochemical incompatibility between the "old" and "new" portions of the system is always present, unless cathodic protection is implemented.

Unfortunately, the current state of knowledge fails to address in a systematic and reliable fashion the issue of electrochemical compatibility and, therefore, to predict with a high degree of confidence the future service life of a repaired concrete structure. The design engineers must do their best in the "durability planning" stage of the project, including an adequate consideration of electrochemical compatibility and related future service-life issues.

Precise in-depth analysis of electrochemical compatibility is very difficult, if not impossible. The difficulties are mainly due to three factors:

- The existing structure has its unique internal environment caused by aging, weathering, and chemical/electrochemical changes and activity (these necessitated the repair). The existing concrete substrates are different from each other in age, quality, service exposure, and condition;
- The application of a repair alters the internal environment. The exterior environment depends largely on the structure's geographical location (temperature, relative humidity, rainfall levels, prevailing winds, and soil types) and the human activity nearby (industrial or traffic-generated pollution). The internal environment that exists within the structural elements is modified by the engineered repair design, which acts as a new boundary between the bulk concrete and the surrounding environment; and
- In a repair system, the internal environment is a moving target, constantly changing due to the existence of the internal transport mechanisms (in addition to the exterior transport described earlier). Water with dissolved salts may be put into movement by temperature and/or pressure gradients. If there is a concentration gradient, dissolved substances can also travel by diffusion through pore water. Finally, ions do migrate in an electric field.

Electrochemical incompatibility is defined as the imbalance in electrochemical potential between different locations of the reinforcing steel because of the dissimilar environments caused by a repair.[10] The dissimilar environments can be related to the differences in physical properties, chemistry, and internal environments. It is widely accepted that electrochemical incompatibility is the driving force for the phenomenon of corrosion reinitiation in repaired structures between the repair and substrate. When reinforced concrete is repaired, some of the chloridecontaminated concrete may be left in place, which ends up in the repair material having a chloride content different to that of the surrounding concrete, in addition to inevitable differences in moisture and oxygen contents. Strong corrosion cells may be established, resulting in spalling of the repair itself or of the surrounding area, a phenomenon often referred to as the "halo" or "ring" effect.

During the condition evaluation phase and durability planning phase, macrocell corrosion caused by electrochemical incompatibility is generally cited as the explanation for corrosion of reinforcing steel in the concrete substrate near a repair area. At the same time, the possibility of microcell corrosion is overlooked, even when it might be a significant part of the overall corrosion activity.[12] This, at least in part, can be explained by the fact that the common measuring techniques essentially detect the presence of macrocell corrosion. Because either microcell or macrocell corrosion (or both) can be the main reasons for corrosion damage in the substrate near repairs, greater attention should be paid to the underlying mechanisms to clarify the following issues:

Table 1:
Durability planning issues and approach[11]

Function and type of structure	Client's basic needs
Performance requirements	Acceptable technical performance Serviceability and safety criteria Importance of continuity of function during repair Accessibility Desirable service life Maintenance strategy
Loads	Dead and live loading
Exterior environmental loads	Water, temperature, and wind effects Aggressive agents and actions
Internal conditions	Cracking, microcracking, other flaws Carbonation, chloride ion content, alkali-aggregate reaction (AAR), sulfate attack, etc. Reinforcement corrosion section loss, debonding
Overall design approach	Basic remediation strategy (do nothing and monitor, provide protection, repair, belt-and-suspender approach)
Evaluation of alternative solutions	Costs Constructability and quality issues Known experience of performance

- Even if macrocell corrosion is detected in repairs (often the case), the total anodic corrosion rate is unknown without measuring the coexisting microcell corrosion; and
- If macrocell corrosion is not detected in repairs, microcell corrosion—which itself can exist and develop after repair—can't be ruled out.

Internal and External Environments

To provide adequate resistance to aggressive actions, it is necessary to foresee how a given repaired structure will deteriorate. This in turn helps yield valuable information on how such deterioration can be prevented or, more realistically, how to ensure a sufficiently slow deterioration process. In other words, the aggressiveness of the existing internal (that is, within the elements or members) and exterior environments, their interaction, and the possible changes caused by the repair should be given comprehensive consideration at the design stage (Table 1). Such analysis is necessary to achieve electrochemical compatibility and fulfil the durability and structural safety requirements of aging concrete infrastructure.

One important goal when performing condition assessment is to define the existing internal environmental conditions (Table 1). These conditions do not necessarily apply to the concrete structure as a whole; rather, they may apply to elements and zones of the structure having radically different local micro-conditions. Hence, the internal environment and corresponding levels of contamination and deterioration can vary significantly, depending on the local exterior environment.[13] For example, the severity of exposure can be very different on the windward side of a pier than on the lee side. Similarly, areas of piers that are subject to saltwater spray, but sheltered from rain, are exposed to a more severe chloride exposure due to fewer washouts.[14]

When internal conditions are evaluated prior to potential repair works, the existing concrete structure is undergoing a certain mass transport activity. The removal of the damaged concrete, repair of reinforcing steel, and application of a repair material alter the existing internal environment and the transport mechanisms. Such changes, as well as their present and future effect on electrochemical behavior and degree of compatibility, need to be analyzed in a project's durability planning phase.

Conclusions

A successful repair project is not permanent - no more in fact than a new construction project. The repair project objectives should be based on rational engineering compatibility analysis, supplemented by knowledge from observations, and enhanced through experience. In the words of famous mathematician Jacob Bronowski, "a good prediction is one which defines its area of uncertainty; a bad prediction ignores it."[15]

To achieve service life expectations on a repair project, compatibility-related issues must be addressed - particularly electrochemistry, shrinkage, creep, mass transport, and aging. However, there is still no reliable accepted design methodology addressing compatibility issues and providing the practicing engineer with guidance towards successful repair.

Among the factors affecting compatibility, electrochemical compatibility is the most complex and critical factor for adequate performance and durability of concrete repair projects. Currently, unless a thorough cathodic protection system is implemented, the risk exists of corrosion recurring and even accelerating due to electrochemical incompatibility between the "old" and "new" phases of the system.

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CASE STUDY

Reprint from CI Magazine, Volume 39, No 11, Page 44-48

A Tale of Two Dams

Parallels and partings in the histories of Oroville and Daniel-Johnson Dams

by Kenneth D. Hansen

The well-publicized failure of the primary service spillway for California's Oroville Dam on the Feather River (Fig. 1) has piqued the interest of the dam building profession worldwide. The total cost of the repairs and reconstruction of the service and emergency spillways has been estimated at $550 million USD.[1] For the damaged main spillway alone, 320,000 yd3 (245,000 m3) of roller-compacted concrete (RCC) will be used to fill in a massive scour hole. It will then be capped with conventional reinforced concrete. For the emergency spillway repairs, it's estimated that about 800,000 yd3 (more than 600,000 m3) of RCC will be needed.[2]

I believe the failure had its origins, at least partially, in the selection of the dam type. For support, I will compare Oroville Dam, which was built by the California Department of Water Resources (DWR), to Daniel-Johnson Dam (originally called Manicougan 5 or simply Manic-5), built by Hydro Québec in Canada (Fig. 2). Both dams produce hydroelectric power (the installed capacity of Oroville Dam and Daniel-Johnson Dam is 819 and 1528 MW, respectively), and both were built at the same time, in the 1960s.

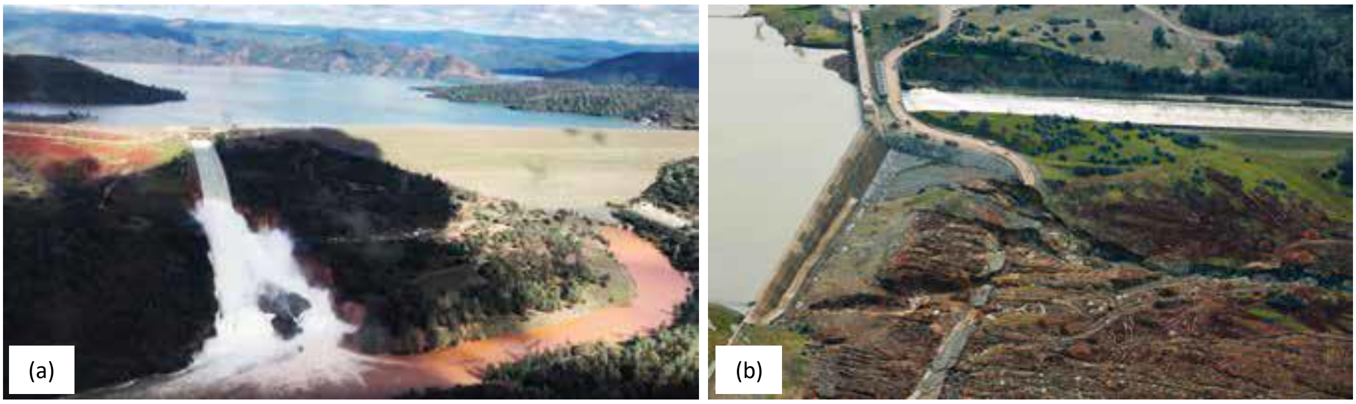

Fig. 1: Oroville Dam, near Oroville, CA: (a) this image, taken from video shot on February 10, 2017, shows water flowing over the damaged main spillway. The emergency spillway wall is visible to the left *(photo courtesy of Josh F.W. Cook/Office of Assemblyman Brian Dahle via AP)*; **(b) after the main spillway was closed, water flowed over the emergency spillway. This photo, taken on February 13, 2017 (after the main spillway was reopened), shows the erosion that nearly undercut the concrete wall** *(photo courtesy of AP Photo/Rich Pedroncelli)*

Fig. 2: Daniel-Johnson Dam and Manic-5 Generating Station in Quebec, Canada *(photo courtesy of Hydro Québec)*

Parallels

The parallels continue beyond function and timing. Both dams are more than 700 ft (213 m) tall and are record holders. Oroville Dam is a 770 ft (235 m) earth embankment dam and is the highest dam in the United States. Daniel-Johnson Dam is a 702 ft (214 m) multiple arch concrete dam and is the highest multiple arch dam in the world.[3-5] Also, to some extent, noted French dam designer Andre Coyne was involved with both projects.

He served as an outside consultant during the early phases of the Oroville project, and his firm, Coyne et Bellier, was the key design consultant for Daniel-Johnson Dam.

Both dams were also built with political implications. According to a recent newspaper article, Edmund G. “Pat” Brown, then Governor of California, as well as his predecessor Goodwin Knight, wanted to bring northern California’s water as far south as the border with Mexico.[6] Although the massive project was quite expensive, the state undertook the project on its own, without federal assistance. This was done in part to avoid restrictions imposed by the National Reclamation Act, which stipulated that no farmer operating an irrigated tract larger than 160 acres (65 ha) (320 acres for a married couple operating the tract) could benefit from water supplied by a project constructed by the U.S. Reclamation Service (later named the U.S. Bureau of Reclamation, or USBR). While much of the water from the Oroville project was to be transferred to greater Los Angeles, some was also promised to the southern reaches of the San Joaquin Valley, which had been operated as large tracts by politically connected farmers.[7]

The state government’s aversion to federal oversight could also have been an attempt to appease populist sentiments. As then-DWR Director Harvey Banks said later, “There is no justification whatsoever for the people to expect the Great White Father in Washington to solve all our problems.”[6] Interestingly, California’s current Governor Jerry Brown (Pat Brown’s son) has asked the federal government for help in paying for the serious damage at Oroville Dam. Since that request, some $22.8 million in federal assistance has been provided for the emergency response.[8]

In Canada, Daniel-Johnson Dam was a symbol of the new Québec nationalism. Hydro Québec was popularly called a “Colossus on the march,” and Robert Bourassa, then-Premier of the Province of Québec, noted that “Manic-5 has become a symbol of our ability to meet the challenge of science and technology.” Further, he said “Manic-5 reflects the image of today’s Québec—dynamic, daring, demanding, and fascinating.”[9] The construction of the dam was closely followed by the public and became a part of the culture of the era. Even though it hasn’t been free of problems, the graceful arches of Daniel-Johnson Dam remain viewed as a national treasure in the province.

Type Studies for Oroville Dam

Feasibility studies for Oroville initially included only concrete dams, due in part to the fact that the five highest U.S. dams at the time were concrete and the site had a competent rock foundation. The types of concrete dams studied included gravity, multiple arch, straight buttress, and arch buttress types.[3] Soon after the studies started, a rockfill dam was added to the dam types to be studied.

The Portland Cement Association (PCA) retained Coyne to visit the site by helicopter and to devise a design for a concrete dam that would be the most cost effective at the Oroville site. His design proposal consisted of an attractive multiple arch dam (Fig. 3). This design would require less concrete, but it would also require more forming than a typical concrete gravity dam. However, labor for building and erecting forms was relatively cheap at that time in history.

Fig. 3: Artist’s rendering of Andre Coyne’s multiple arch concrete dam design for the Oroville site (based on Reference 10)

Because the interaction of the various elements of Coyne’s multiple arch design was so complex, the DWR retained Jerry Raphael of the University of California, Berkeley to build and perform measurements on a large plaster-celite model of the design. Raphael had extensive experience with concrete dam design as he had worked at the USBR for 15 years before joining the staff at Cal Berkeley.[10] It’s of interest to note that one of the graduate students who worked on this model study was Edward (Ed) Wilson, who later became renowned for writing the first widely accepted finite element computer package for structural analysis (SAP).[11]

Also as part of the DWR investigation of the concrete dam alternatives, the DWR evaluated gold mine tailings located approximately 8 miles (13 km) downstream on the Feather River for suitability as concrete aggregates. Led by Lewis H. Tuthill (who subsequently served as ACI President in 1961), the investigators determined that the granite and schist cobbles in the tailings were of high quality and suitable for concrete production.[3]

However, the materials investigators also found that fine-grained soils that could be sieved from the tailings were suitable for use in an impermeable core for an earth embankment dam.[3,11] Lack of clay for an impermeable core at rock foundation sites that can accommodate a high dam would normally eliminate the consideration of an earth embankment dam at this time.

Soils engineering (now called geotechnical engineering) was just hitting its prime in the late 1950s, and I'm sure that engineers in this discipline were itching to make a statement by designing a very high earth dam. With the discovery of the fine-grained material, it appears that DWR's decision makers began to lean toward an earth embankment, even though this dam type was not on the original list of types under study.

Citing lack of precedence for a high multiple arch dam solution (Bartlett Dam in Arizona, at 305 ft [93 m], was then the highest multiple arch in the United States)[12] and the estimated cost of extensive foundation preparation for the concrete multiple arch alternative, DWR decided in November 1958 to design an earth embankment dam for the Oroville site.[3] They did not even wait for the results of Raphael's model studies, which were released in 1960.[10]

I find it dubious that the earth dam was selected to avoid extensive foundation preparation. First, the footprint for the multiple arch dam was less than 20% of that for the earth embankment, which also needed some foundation improvement. Secondly, the design of the multiple arch dam had evolved to include a large central arch that would have spanned the Feather River gorge, and thus no buttresses would apply a concentrated load in this troublesome area.

Rather, it's apparent to me that the staff of the DWR (only a few years old at the time) did not have confidence in their expertise to design a high multiple arch concrete dam. This was despite Raphael's study, which aimed at determining limiting concrete stress values. Universities in the United States simply did not (and still do not) produce concrete dam engineers (or, in fact, concrete spillway designers). Concrete dam engineers were thus mostly structural engineers who learned from experience at major dam consulting firms or the USBR. However, it appears that most of the senior staff of DWR and its consultants had studied soils engineering and thus would have favored an earthfill dam. Oroville was a geotechnical engineer's dream, as it contained 10 different zones of earthen materials and was curved upstream.[3,13]

Type Selection for Daniel-Johnson Dam

Hydro Québec's staff was more daring in their selection of dam type. Coyne's multiple arch concrete design was chosen in mid-1960, despite the lack of precedence for this type of dam in Canada. Coyne et Bellier worked as the dam design consultant to Montreal, QC, Canada-based Surveyor, Nenninger, and Chenevert Inc. (now SNC-Lavalin).

The confidence of Hydro Québec's design team may have been boosted by the linguistic link with France's most noted concrete dam designer. However, a more substantial reason for going with Coyne's concrete dam design had to do with electricity production. To meet electrical demand in Québec Province, the reservoir water had to reach a specified level by a certain date. Because the reservoir was very large, filling had to start early.[13]

A rockfill alternative would have required more than 30 million yd3 (23 million m3) of material. Hydro Québec determined that it was not feasible to produce this much material and meet the time schedule, considering that construction was limited by cold weather. The designers also figured that it was also not feasible to construct a concrete gravity dam and meet the time schedule. In contrast, a multiple arch dam could meet the schedule because filling could start earlier (even before completion) than it could for the alternative rockfill dam design. The multiple arch provided other advantages. It would require less concrete (a significant factor given the high prices of concrete at this remote site), which led to a lower estimated cost over competitive dam types. In addition, the large central arch simplified spanning the site's deep river channel, which extended 150 ft (46 m) below the river bed to competent rock.[4,13]

Design Revision for Oroville Dam

A geological investigation in the bed of the Feather River uncovered the irregular and narrow incised nature of scoured potholes in the foundation of the center of the earth dam. This caused DWR's Consulting Board to recommend that a concrete core block be built in this area. The core block served to eliminate abrupt changes in the foundation profile for the impervious core (Zone 1) material as it crossed the steep-sided river channel (refer to Fig. 4). This reduced the possibility of internal shearing of the earth core material due to differential settlement.

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The concrete core block was constructed in 1963. It required the placement of 293,000 yd3 (224,000 m3) of lean mass concrete using 6 in. (150 mm) maximum size aggregate. The main concrete section was up to 63 ft (19 m) above the channel bottom. A 50 ft (15 m) high, triangular-shaped parapet was placed on the upstream side of the block, extending over a length of 880 ft (268 m). The parapet served to protect adjacent embankment fill from damage during the 1963-1964 winter runoff. In addition, a 128 ft (39 m) high cofferdam, requiring 252,000 yd3 (193,000 m3) of concrete, was built on the rock foundation.[13,14] If a concrete dam had been built at the Oroville site, it would not have required such a high cofferdam because the concrete could have been overtopped during construction without failure. In my literature search, I found no mention of the structural design of the Oroville spillways. I was able to find that their combined capacity was 624,000 ft3/s (17,700 m3/s) with the main service spillway having a capacity of 150,000 ft3/s (4250 m3/s). The service spillway was described as having eight submerged radial gates and a lined chute for flood control. The crest control for the emergency spillway was described as a combination of an ungated ogee and a broad-crested weir.

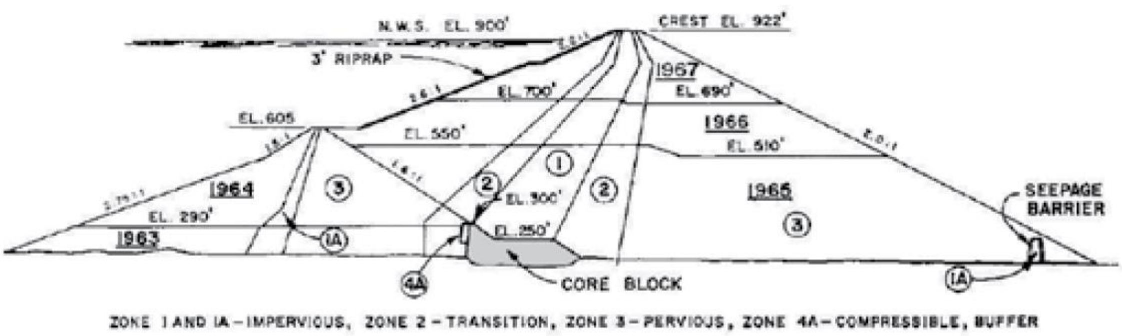

Fig. 4: Section through Oroville Dam showing relative location of core block (Fig. 1 in Reference 14)

Considering the recent developments, it is readily apparent that less attention was paid to the design of the spillways than the embankment. Both the main spillway and emergency spillway were poorly designed. The main spillway's physical flaws have been well chronicled.[15] It seems the spillway designers did not have a full appreciation that they were designing the highest head spillway at 703 ft (241 m) and the highest unit discharge spillway ever in the United States. They were breaking new ground with respect to the amount of energy and high-velocity flow that needed to be accommodated in Oroville's main service spillway.

The emergency spillway, which consisted of a series of 45 ft (14 m) wide concrete gravity monoliths, is also considered to have flaws. The design included no provision for energy dissipation to prevent erosion of the earth embankment downstream, nor included a cutoff wall to prevent head-cutting. While I could find no statement in the public record, I understand this feature was not included in the emergency spillway because the designers felt this spillway had very little chance of operating, and they wanted to save money.

Comparative Cost of the Two Dams at the Oroville Site

DWR made a comparative cost estimate between the earth and concrete dams for Oroville. One of the reasons for selecting the earthfill gravity dam was the estimated high cost of extensive foundation treatment for the multiple arch dam. As discussed previously, I feel this assertion is questionable. After the designers decided to add the concrete core block under the impervious core material, the cost advantage due to foundation improvement was reduced, if not eliminated.
Around the time of construction of Oroville Dam, unit price bid tabulations for other large dam projects in the United States showed large volumes of concrete could be placed at low in-place cost, including cement[15]:

- Glen Canyon Dam, Arizona (1960 median year of construction), using 4,770,000 yd3 of concrete bid at $12.34/yd3 (3,647,000 m3 at $16.14/m3);
- Dworshak Dam, Idaho (1970), using 6,600,000 yd3 of concrete bid at $11.65/yd3 (5,046,000 m3 at $15.24/m3); and
- Libby Dam, Montana (1970), using 3,660,000 yd3 of concrete bid at $14.49/yd3 (2,798,000 m3 at $18.95/m3).

The concrete multiple arch dam would have required an additional cost for forming and probably a little more cement due to higher stresses. In addition, placement rates for concrete in the multiple arch would have been lower than for these large concrete arch or gravity dams. While DWR could not have been aware of this relevant cost data at the time of their dam type decision, it is reasonable to assume their cost estimate for concrete in the multiple arch alternative would be high and thus not favor the concrete dam alternative.

If the multiple arch concrete dam alternate had been built at the Oroville site, I believe the estimated 4.2 million yd3 (3.2 million m3) of concrete could have been placed in 2 years' less time than building the earth dam. It seems that Governor Pat Brown, who was pushing for completion of Oroville Dam, would have liked this if he had been aware of this fact. It is, however, a certainty that the cost benefits of completing the concrete dam 2 years quicker than the earth dam were not considered. While earlier completion provides earlier water supply, flood control, recreation, and fish benefits, the greatest economic benefits are in electrical production.

At the time of the completion of Oroville Dam in 1968, the cost of electricity in California was approximately 2.4 cents/kWh. It is now about 16.3 cents/kWh. If we assume a conservative use factor of 0.6, the dam's 819 MW capacity could have generated more than $200 million income in 1968 dollars. If this factor had been considered, the total cost would have easily favored Coyne's multiple arch concrete design.

Construction of the Dams

Construction of both dams proceeded concurrently. Placement of 80,700,000 yd3 (61,700,000 m3) of earth fill in Oroville Dam and 2,880,000 yd3 (2,200,000 m3) of concrete in Daniel-Johnson Dam started in 1962 and ended in 1967. Construction at each site was not without its problems. As expected, at Daniel-Johnson Dam, concrete could not be placed in cold weather, which limited concrete placement to 140 days per year on average. Oroville was plagued with strikes and worker fatalities. The partially completed dam also withstood one major flood - a record flow of approximately 250,000 ft3/s (7080 m3/s) in December 1964.[3]

In Oroville Dam, the greatest earth fill placement rate was 25.5 million yd3 (19.5 million m3) in 1966.[2] For Daniel-Johnson Dam, the highest volume of concrete placed in a single month of 26 working days was just over 137,500 yd3 (105,130 m3).[3] In 1966, concrete placement averaged nearly 5300 yd3 (4050 m3) per day.[16]

Performance of the Dams

Both dams have had some performance issues since they were put in service nearly 50 years ago. Most of the performance issues for Daniel-Johnson Dam are related to extreme temperature variations, as temperatures there can dip as low as −58°F (−50°C).[17] The dam has exhibited some cracking due to thermal differentials and shrinkage as well as damage related to cyclic freezing and thawing. The latter problem was addressed by the 2012 application of a polyvinyl chloride geomembrane to the dam's upstream face.[18] The membrane replaced the dam's original asphalt coating and mitigated seepage through the section.[18]

At Oroville Dam, initial problems included cracking of the unreinforced concrete core block parapet due to pressures from the earth fill applied on the upstream side. Although most of the dam's piezometer tubes were broken at the location where the tubes were bundled together in vertical risers through the central core, investigators concluded that the safety of the dam was not affected. In fact, the dam survived well the nearby Oroville earthquake (M = 5.7) in 1975.[3]

Over the years, cracks in the primary service spillway at Oroville have required repairs, and the spillway has required continuous maintenance of its transverse joints. However, none of the performance problems at Oroville were considered major until the recent service spillway failure.

The Spillway Situations at both Dams

The Forensic Review Team selected to investigate the cause of the major problem with the Oroville Dam service spillway has listed 24 potential design and maintenance problems.[16] While they have not issued their final report at the time of this writing, it's likely that they will conclude that the spillway failure was caused by a combination of poor design and construction.

Rather than a single primary spillway, Daniel-Johnson Dam has two spillways, one located on each abutment. I could uncover no published reports on the design of the spillways, nor any problems associated with operation of these spillways. It seems that the Canadian and French concrete dam designers were more experienced in the proper design of spillways.

Conclusions

I believe that if the multiple-arch concrete dam had been selected for the Oroville site, it would have been less costly and quicker to build than the earth embankment dam that was constructed. The concrete dam also would have become a tourist attraction that would have rivaled Hoover Dam. Although the state put forth plans to build an amphitheater and restaurant at the site and construct a tourist train to the lake, it never delivered on those plans.[19] The state might have followed through on their promises if it had constructed a dramatic multiple arch dam.

Still, the present situation with the main service spillway probably would have remained the same. In both the earth and concrete dam solutions, the main spillway is sited in basically the same location. Unless the concrete dam designers had provided a better spillway design, the disastrous results could still have occurred. The design of the main spillway associated with Coyne's multiple arch proposal is unknown. However, from Fig. 3, it appears wider than what was actually built for Oroville. The details of their spillway are also unknown, but could not have been worse.

What DWR needed was someone with more expertise in the design and construction of long, high-capacity concrete chute spillways. Unfortunately, the DWR staff and consultants lacked the necessary expertise, and simple provincialism apparently prevented them from looking outside the state for help.

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