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Autogenous Crack Control during Construction Phases of MOSE Venice Dams

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Abstract. The design of concrete structures exposed to severe environmental attack, like in marine environment, requires serious attention for concrete durability. Early age cracking due to autogenous deformations can be detrimental to the performance of tidal structures. The study of the structural effects of hydration heat and rheological behaviour of a set of huge concrete structures of the Mobile Venice Dams known with the MOSE acronym (Experimental Electromechanical Module) is presented in this paper. Together with other measures such as coastal reinforcement, the raising of quaysides, and the paving and improvement of the lagoon, MOSE is designed to protect Venice and the lagoon from tides of up to 3 meters. Construction began simultaneously in 2003 at all three lagoon inlets, and the project has been completed in 2014. Floods have caused damage since ancient times and have become more frequent and intense as a result of the combined effect of eustatism (a rise in sea level) and subsidence (a drop in land level) caused by natural and man-induced phenomena. Nowadays, towns and villages in the lagoon are about 23 cm lower with respect to the water level than at the beginning of the 1900s. Each year, floods can cause serious problems for the inhabitants as well as deterioration of architecture, urban structures and the ecosystem. Over the entire lagoon area, there is also a constant risk of a catastrophic event such as that of 4 November 1966, when a tide of 194 cm submerged Venice, Chioggia and the other built-up areas.

1. Introduction

The protection of Venice from flooding tides is obtained by isolating the lagoon from the sea through four mobile barriers crossing the inlets of the lagoon for a total extension of about 1600 m.

Any barrier consists of several steel flap-gates. In normal tidal conditions, the gates are full of water and resting on their housing caissons. When High Water ("Acqua Alta") Forecast predicts tide/storm levels higher than the safeguarding level, the gates are partially emptied by the introduction of compressed air and raised (see Figure 1) to form a barrier; the connection with the foundation is ensured through steel units named hinge-connectors.

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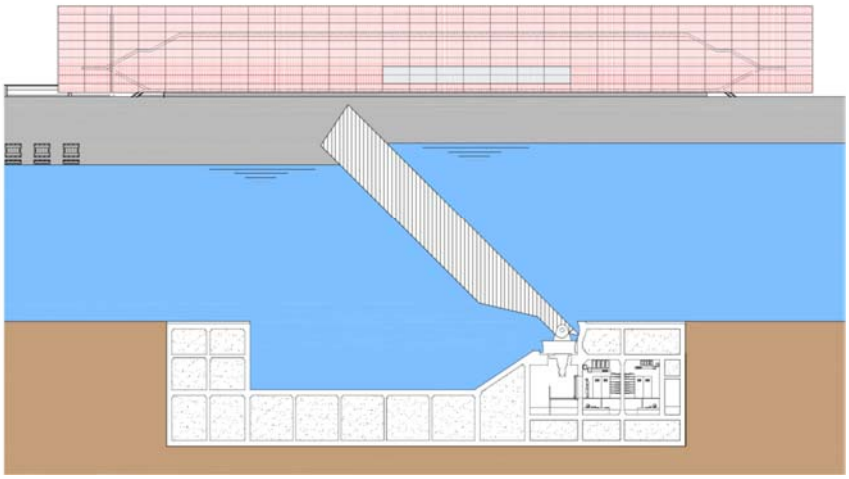


Figure 1. Cross section of the barrier with raised gate

The row of gates of each barrier is supported by 6-7 boxes made of precast reinforced concrete and known as gate caissons. These elements cover the entire width of the inlets and are completely buried, as shown in Figure 2.

The closing structures of the barrier function as connection between the intermediate immersed section (360-420 m long and everywhere below the seabed) and the banks; they consist of two caissons emerging from the sea water level, each composed of a high precast cellular structure (up to 27.5 m) that will be topped off on site, after the completion of sinking and ballasting operations, with the construction of additional floors.

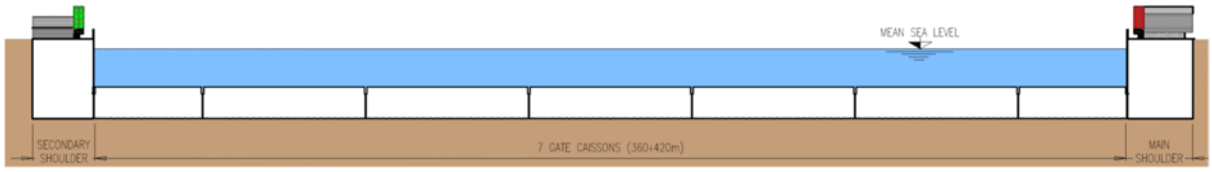


Figure 2. Typical longitudinal section of the barrier

All the cellular caissons have been prefabricated and assembled with the typical technique of the "immersed tunnels". More than a hundred road and rail tunnels have been built worldwide with this technique and others are being planned, but Venice caissons are unique in their features and functionality.

The barrier is designed for a service life of 100 years. The choice of materials and corrosion protection systems required careful consideration to fulfil the requirements of water-tightness and durability, to control the development of cracking phenomena and assess the behaviour under cyclic loading (fatigue).

2. Gate caissons description and construction technique

The primary function of the gate caissons is to allocate and preserve the power and service plants of the gates. The dimensions of the structures depend largely on the size of the gates and water depth of the channels. The largest ones are 60 m long, 11.5 m high and 50 m wide, for a weight of about 20.000 t.

All the barrier caissons have a multicellular structure, working as a membrane-shell structural system. The gate caisson is a cellular structure characterized by 4-5 horizontal slabs and a grid of vertical partitions spaced about 5 m in both directions (average thickness of about 40 cm). Several reinforced

areas have been designed to resist the application of concentrated loads of great intensity (ie. the gate connections). Inside the caissons, most of the spaces are ballast compartments, filled with water and heavy ballast (concrete and/or sand).

The concrete volume used to build the overall precast caissons and the 4 barriers was about 220.000 m^3, with an average steel-to-concrete ratio of about 300 kg per cubic meter of reinforced concrete. In particularly elaborate joints, this amount reaches 500 kg/m^3. Where the risk of water and air penetration in the concrete cover was considered significant, a percentage of about 15-20% of the reinforcement was in stainless steel AISI 316L.

When the caissons of each barrier were completely built and equipped, they were floated and transported above the special trench prepared in the related inlet with the assistance of an installation barge, then sunk in their final position and permanently ballasted. Because of the construction phases involving floating conditions, the nominal weights of the structural elements were subject to a strict control procedure, obtaining a final weight variation with respect to the design values limited to ±1%.

The construction method is based on the prefabrication on site of all the barrier elements (caissons, gates and hinge-connector units) in dedicated areas, away from the bed of the channels to limit interferences with natural environmental and navigation. Temporary dry docks (see figure 3) were built in the future refuge harbours of some inlets or on the dryland. The caissons built on the embankment have the peculiarity to be moved over rails by means of a trolley system, from the precast area to the Syncrolift® platform, where each of them will be lowered and floated in water. The construction of the caissons in the dry docks was carried out working 9-10 m below sea level, in dry conditions (see Figure 4a). At the end of prefabrication, the basin was flooded again to put the caissons in flotation.



a) b) c)

Figure 3. Precast yards. (a) Lido Treporti; (b) Malamocco embankment; (c) Chioggia.

Different contractors have been involved in the construction of the caissons. Each contractor made specific choices in relation to the concrete mix, formwork and volume, sequence of castings and management of the precast yard, based on extensive previous experience. However, the construction of the caissons of the different inlets was characterized by very similar methods.

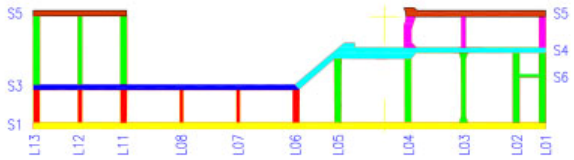
Concrete cast for each batch varies between 100 and 450 m^3, except for the bottom slab, where about 1000 m^3 have been cast in a single pour (see Figure 4b). Based on the size and complexity of the structure, several casting phases have been required to complete a caisson (from 15 for the smallest caissons to 23 for the bigger ones). The horizontal joints coincided with major horizontal elements, while the vertical ones are derived from a planimetric checkerboard scheme of 3x3 fields, as shown in Table 1. In the transverse direction, fields are positioned at the intersection between the high side wings and the intermediate low body, as shown in Figure 4 (b).

Each caisson was completed within 6-9 months; to limit the overall time of prefabrication within about 2 years, at least 2-3 elements had to be constructed in parallel. To reduce part of the deformations due to shrinkage, the metal elements that had to guarantee high precision, such as the fixed parts of the hinge-connectors, were installed on the caissons a few months after the last casting. These elements have been post-installed compensating the potential prefabrication errors.

Initially, on the basis of other international experiences for cellular caissons, it had been considered the use of permanent prestressing to limit cracking of the hardened concrete. This solution, given the aggressive environmental conditions, was abandoned for two reasons:

1. Sophisticated and demanding technologies (like injection under vacuum) were to be applied on a large scale without having the possibility to access to the tendons anchorages once the caissons were launched. The important aspect of durability would have remained unsolved.
2. Once the caissons would have been installed in the barrier, it would not have been possible to carry out inspections and/or maintenance operations. It would be therefore essential to provide (with reference to the EC2 [1]) a minimum reinforcement to guarantee ductile behaviour under frequent load combinations. However, the hydrostatic head is a substantially permanent load and it is responsible for most of the stress. Therefore, important ordinary reinforcement should be provided besides prestressing leading to an uneconomic solution.





a) b)

Figure 4. Construction phases. Gate caisson nearing completion (a) Horizontal joints example (b).

The use of temporary prestressing has also been evaluated. Nevertheless, even this solution has been abandoned because of the heavy construction constraints that would have resulted, like:

1. limited room for the insertion of the casting pipes;
2. difficulties in housing the cable anchorages, causing changes on the reinforcement spacing;
3. the difficulty to accommodate the intermediate anchors in construction joints;
4. important interference among the predalles lattice in slabs realized using collaborating predalles.

It was then developed the r.c. solution, with the insertion of some additional requirements, aimed at ensuring the durability of the caisson for a lifetime of 100 years, which are essentially addressed at reducing the concrete permeability as requested by [1]:

1. use of concrete with reduced hydration heat generation;
2. maximum water/cement ratio = 0.45;
3. maximum cement content + mineral additives = 550 kg/m^3;
4. reduced values of hygrometric shrinkage in the first 30 days (< 20 vm/m after 1 day, < 100 vm/m after 7 days and < 300 vm/m after 28 days);
5. use of high-fluidity concrete, with high compactness, and thus lower permeability;
6. seven days curing during which the concrete castings should be kept at a controlled temperature and humidity to facilitate the hydration and the proper maturation;
7. use of appropriate concrete cover for the reinforcement (variable between 4 and 5 cm) and stainless steel in the most critical areas;
8. use of concrete with low permeability to chlorides according to ASTM C1202-05 [2].

3. Thermomechanical numerical analyses

To ensure adequate durability, maximum attention has been paid to the cracking problem. Because of the intrinsic behaviour of concrete, it is almost impossible to prevent cracking, but the size and location of cracks can be limited and controlled to grant 100-year service life.

A series of numerical analysis has been performed with the aim of evaluating the cracking pattern during the construction phases before caissons immersion, in order to optimize the reinforcement design so that cracks opening, where present, would not exceed the predetermined limits.

Finite element models able to perform thermomechanical analysis have been developed. In a preliminary analysis, the development of heat due to hydration of cement, its diffusion in space and the consequent trend of temperature inside the structure's elements have been obtained. In a second part of the analysis, the effects of thermal deformations and the rheological behaviour of the materials have been studied during the construction phases.

In the structural model, multi-layered three-dimensional shell elements have been used, implementing smeared cracking and reinforcement. The solution simulates the casting stages sequence by activating in succession the parts of the structure following the progress of their realization.

3.1. Preliminary analyses

For each concrete foundation caisson, 10 to 15 types of structural elements have been identified. Each structural element has been divided into 1x1m portions, characterized by different thicknesses and thermal boundary conditions. Each 1x1m portion has been divided in several layers in the thickness direction, in order to take into account, the temperature and mechanical properties development.

It has been assumed that the heat dispersion could only take place through the side faces ($1m^2$), neglecting the heat flows along the contact edges between adjacent elements. It is therefore assumed that each structural portion under analysis is surrounded by areas with similar heat development in time and that the integral of heat flow exchange between adjacent concrete bodies is null. It has also been assumed that moulds were removed 3 days after casting. The heat exchange, after moulds removal, takes place directly with the air, losing the insulating effect of the mould.

The results obtained from the preliminary analysis in terms of imposed deformations (thermal + shrinkage) and variations of concrete properties (Young modulus, tensile and compressive strength, creep) in function of the degree or hydration have been used as input parameters for the second part of the analyses.

3.2. Construction phases analyses

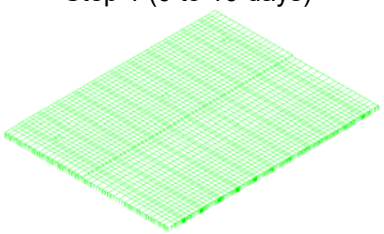
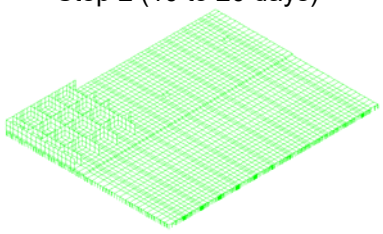
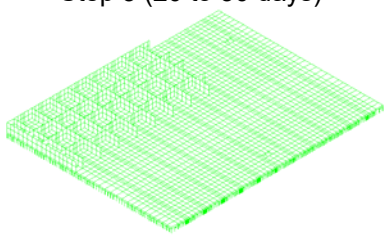
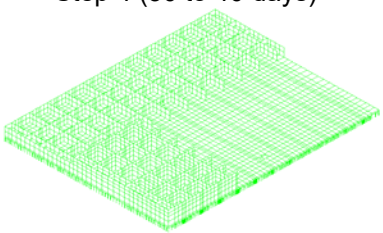
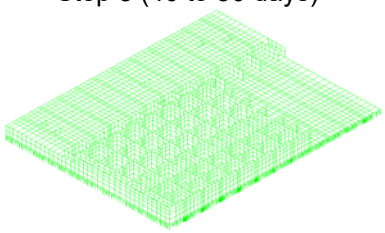
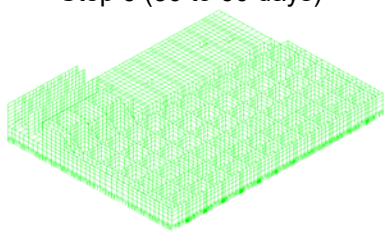
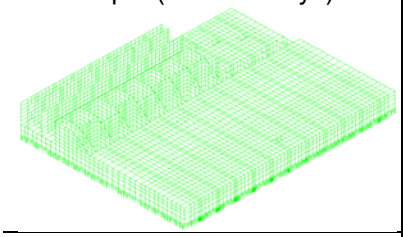
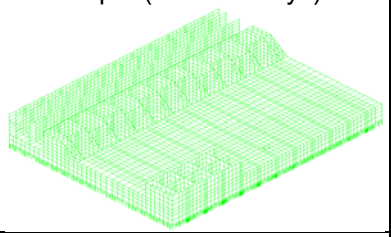
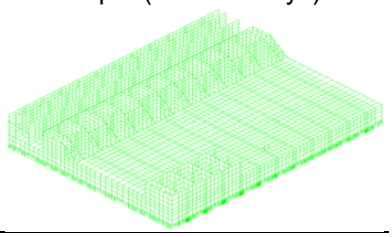
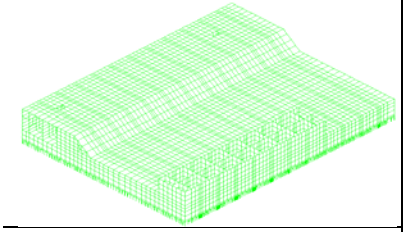
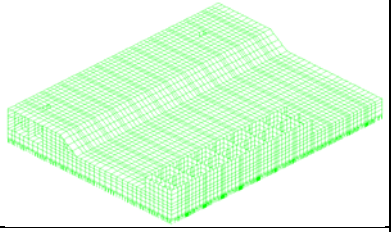
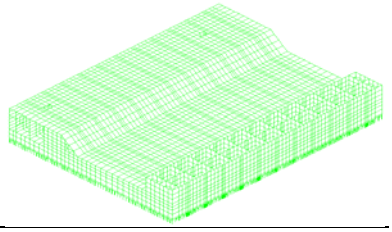
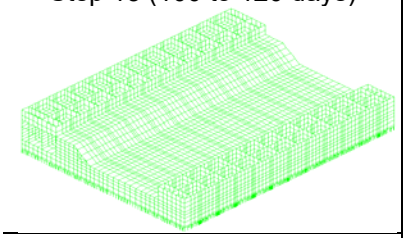
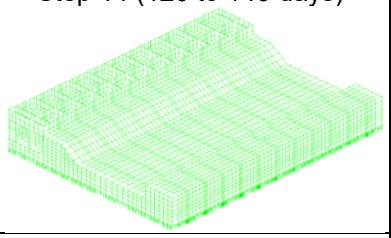
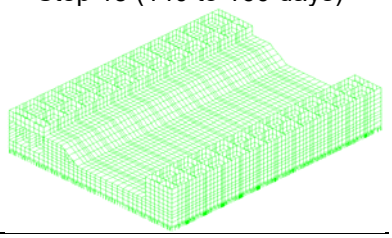
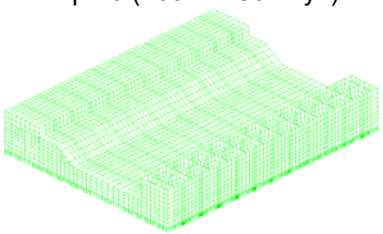
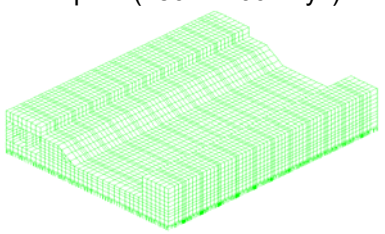
An example of a typical construction phase is reported in Table 1. The physical phenomena considered in the analyses are described in the following paragraphs.

4. Physical phenomena modelled in the analyses

4.1 Hydration heat

The chemical reactions of concrete hydration are all esotermic. Hydration process is deeply influenced by mix design, cement quantity in concrete, water/cement ratio, ambient temperature, cement grinding and the level of the reaction. The physical model used in the present work is based on the theory developed in [3-9].

Table 1. Construction phases

Step 1 (0 to 10 days) 	Step 2 (10 to 20 days) 	Step 3 (20 to 30 days) 
Step 4 (30 to 40 days) 	Step 5 (40 to 50 days) 	Step 6 (50 to 60 days) 
Step 7 (60 to 70 days) 	Step 8 (70 to 73 days) 	Step 9 (73 to 80 days) 
Step 10 (80 to 83 days) 	Step 11 (83 to 90 days) 	Step 12 (90 to 100 days) 
Step 13 (100 to 120 days) 	Step 14 (120 to 140 days) 	Step 15 (140 to 160 days) 
Step 16 (160 to 180 days) 	Step 17 (180 to 400 days) 	

Concrete heat production is related do a fictitious variable *r* called "degree of reaction" as shown in equation (1)

$$r = \frac{Q(t,r)}{Q_{\max}} \tag{1}$$

where Q_{max} is the total hydration heat produced per unit volume until the end of the hydration process and $Q(t,r)$ is the hydration heat produced from the beginning of the reaction at time *t*, that can be written as follows:

$$Q(t,r) = \int_0^t q_r(r) \cdot q_T(T) d\tau = \int_0^t q_{\max} \cdot q_{r,norm}(r) \cdot q_T(T) d\tau \tag{2}$$

where : $q_{\max}$ is the maximum value of the heat power as a function of the degree of reaction $q_r(r)$

$q_{r,norm}(r)$ is $q_r(r)$ normalized to 1

$q_T(T) = \exp\left(-\frac{c_a(T,r)}{T+273}\right)$ is the heat power as a function of the temperature

$c_a(T,r)$ is the Arrhenius constant.

4.2 Heat propagation and diffusion

The specific heat *C* or thermal capacity per unit volume of concrete and its thermal conductivity *K* are modeled according to [10-16] and considered variable in time during hydration.

$$C(r) = C_{r=1}(1+\alpha-\alpha \cdot r) \qquad K(r) = K_{r=1}(1+\beta-\beta \cdot r) \tag{3}$$

Where $0.1 < \alpha < 0.2$ and $0.2 < \beta < 0.3$ are derived from literature and *r* is the degree of reaction.

The heat transfer between concrete casting and the atmosphere takes place involving the three phenomena of conduction, convection and irradiation. In the present work it has been modelled according to [15-18] using a simplified approach based on an equivalent thermal conductivity Ke [J/(s m^2 °K)], whose values depends on the environmental conditions.

4.3 Concrete mechanical properties variation during hydration

Mechanical properties of concrete deeply change during the hydration process as the material changes from fluid to solid state. In the computational model, concrete is always seen as a solid material, whose mechanical properties are very low at the beginning of the hydration reaction and rise while the reaction takes place.

The variation of compressive strength, tensile strength and modulus of elasticity, has been taken into account in relation to the degree of reaction as shown by equation (4) (according to the formulations proposed in [4-7]) obtaining results in accordance with MC90 [19], Carino [20], and De Shutter [21].

$$\frac{f(r)}{f(r=1)} = \lambda \cdot r^a + (1-\lambda) \cdot r^\alpha \geq c \tag{4}$$

Where λ, *a*, α and *c* are parameters that assume different values when used for compressive strength, tensile strength and modulus of elasticity and are all derived from laboratory tests on young hardening specimens.

4.4 Creep and Shrinkage

Shrinkage has been modelled with special attention. As drying shrinkage is sensibly influenced by the hydraulic radius of the member, it can be greater in the regions of the casting near the outer surface than in the inner core. Moreover, massive bodies cannot be described by a single hydraulic radius, they should be divided into different zones for which different shrinkage functions have to be calculated. The dimensions of these zones should be chosen in order to have a smooth variation of shrinkage values inside the model.

The knowledge of the time dependent mechanical behaviour of the concrete is extremely important in order to properly determine the stresses during the hardening process. Due to the creep effect, the stress pattern and its evolution in time can deeply change in comparison to what would be obtained neglecting the viscoelastic behaviour.

Creep and shrinkage have been studied experimentally by subjecting cylindrical samples to increasing levels of compression stress at very young age (2 days, 3 days and 7 days). The laboratory tests have been numerically reproduced taking account of hydraulic radius variation.

5. Comparison between numerical results and measured cracking phenomena

A limit for the maximum crack opening of 0.15mm on the exterior surface has been chosen, assuming that the classic bottleneck shape would reduce the crack amplitude of at least 30% near the bar.

The numerical analyses lead to the individuation of the regions more likely to crack and gave the possibility to integrate the reinforcement, wherever necessary, in order to reduce the opening to the desired values. Figure 1(a) shows the cracks orientation as predicted by the fem model on a wall of one of the caissons, whereas Figure 1(b) is a picture of the same wall taken on site (the cracks have been put in evidence using green lines).

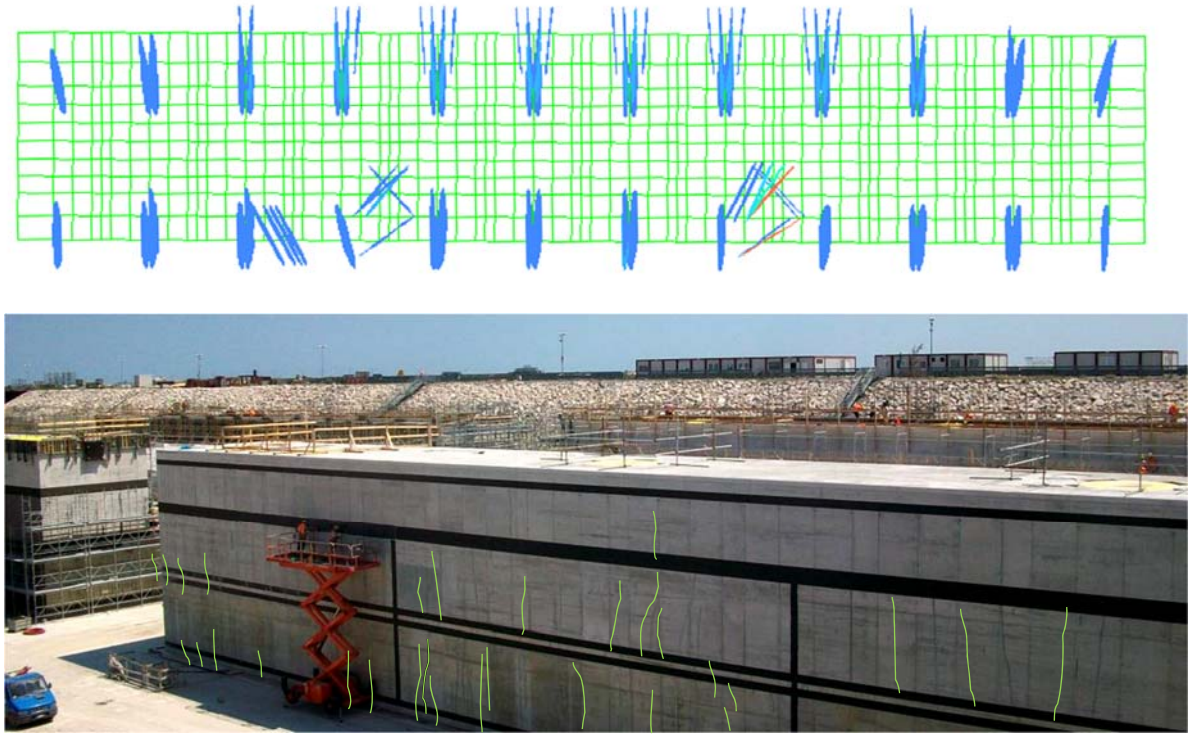


Figure 5. Crack direction and position in numerical prevision vs. real crack pattern

It should be noted that for such a highly hyperstatic structure (length of walls up to 60 m), cast in multiple stages carried out over a long construction period (16 casting stages and a construction time of

about 6 months) constantly exposed to thermal excursions, the development of the cracking phenomenon is considered inevitable, and therefore awaited. In fact, all the caissons have shown the presence of cracks, although with localization, progression and intensity different from site to site. Crack's opening values have been lower that the design values.

6. Conclusions

The structural system of gate caissons described in the present document, alongside with the gates, is one of the most important element of MOSE, as well as a significant engineering work. In fact, caissons play an important role in providing a solid base to one of the most modern dam systems realized in recent years, designed to defend lagoon cities, like Venice and Chioggia, from high water caused by a tidal wave which is the largest in the Mediterranean Sea.

For these reasons, design and manufacturing choices for such a complex infrastructure have been adapted to multiple factors, variables and singularities that are peculiar to the site and are difficult to find elsewhere. In particular, specific attention has been paid to the aggressive environmental conditions (sea water) in which the workpiece is placed to ensure adequate durability to the structure.

The analysis of the cracking phenomenon was therefore considered of significant importance, especially during the construction phases and in the following months. The caissons have been constructed in designated areas of the site located at a distance from the final installation point, and then transported by floating up to the trenches of pose. The potential formation of cracks during the construction phases would have resulted in significant problems during the transport in water and subsequent sinking of the caissons.

The non-linear analysis carried out taking into account the specific properties of the used concrete mixtures and the constructive steps agreed with the different construction companies helped to identify areas potentially subjected to significant cracks because of endogenous self-restrained strains. Thanks to this analysis, reinforcement, previously dimensioned on the basis of the design loads combinations, has been integrated where needed in the areas in which the cracking deformation exceeded 0.12-0.15E-3. Maximum crack opening of 0.15mm on the exterior surface has been considered during construction.

Experimental data show that only 5-10% of detected cracks had a width greater than 0.15 mm. The SCC mixtures, involving the use of small size inert, have detected a greater fragility as compared to standard mixtures. This range of cracks opening values is in line with the design choice, considering that the characteristic values have the probability of 5% to be exceeded.

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