MODELLING OF ALKALI-AGGREGATE REACTION IN CONCRETE PILE CAPS STRUCTURES

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Abstract

Many concrete reinforced structures throughout the world are suffering from deteriorations induced by alkali-aggregate reaction (AAR), which directly impacts on their durability and serviceability. The concrete expansion produced by AAR leads to cracks throughout the structure. In this direction, this paper presents a methodology for assessing the effects and use of back analysis methods for the interpretation of the behavior of concrete by means of a predictive mathematical model based on finite element method. This approach allows simulating orthotropic swelling of concrete subjected to AAR, including effects of the reinforcement presence, the 3D stress state and humidity. A real case study will be presented, showing a set composed by column, pile caps and piles of a building foundation structure that suffered the effect of AAR.

Keywords: steel roof structure, mathematical model, orthotropic swelling, confinement stress, restrain due reinforcement.

1 INTRODUCTION

To readjust a building so that it may go through any changes in its structure, it is essential to investigate with the purpose of obtaining the structural capacity of its elements. *Edifício 10,* which belongs to the Administrative Complex of a big company in Brazil, is currently out of order, and will be redesigned for its commercial use after the inclusion of two new floors. However, during the course of the foundation blocks investigation (Figure 1) it was detected some pathological manifestations, such as cracks, possibly caused by expansions due to AAR phenomenon.

AAR is a chemical reaction between the concrete alkalis– predominantly sodium (Na) and potassium (K) - with the reactive minerals present in the aggregates. The water is an essential factor for this reaction to occur, since water is needed to form the expansive gel that fills the concrete pores, resulting on a deleterious expansion.

The symptoms of this reaction might take decades to be noticed, considering that AAR, generally, is a slow process. However, some external factors may intensify the reaction, such as temperature, an external compression the structure might be submitted to, and the availability of external alkalis. The last one could increase the ionic concentration.

Therefrom, were held: inspections, extraction and specific testing on concrete cores (to characterize a possible AAR) and a computer model simulation of the reaction and behavior of the structure using the ANSYS software.

These analyses will provide support to determine the morphology of the pathological manifestation and shall indicate which actions should be taken from that point.

2 INVESTIGATION

2.1 Visual inspection

Visual inspection of structures is an essential method to monitor behavior and evolution of any pathology that may be present or arise on its elements. The inspection is conducted according to specific criteria, being applied: simple documentation and registering data, routine or special investigation. Such criteria differ in frequency and minuteness level. The first of which is recommended for the period right after the conclusion of civil works, to gather "as built" documents and indicate the construction technologies applied. The second, which should happen within 1 to 2 years intervals, is planned to identify anomalies in elements, and the third and last, similar to the second, is, however, performed on complex/large scale structures, or if any unsafe or unexpected behavior of the building is noticed.

In the studied case, the building's foundation blocks were inspected and mapped, showing its current state of conservation (Figures 2 and 3).

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2.2 Ultrasound tests

The ultrasound is a device equipped with a data logger, a transmitter and a receiver of ultrasonic pulses. It is used to measure the pulse velocity across an element section, from transmitter to receiver, and determine the material's quality, considering that the time variable, for the pulse to overpass the path, depends on characteristics such as elasticity and density.

This means allows analyzing the quality of the concrete and estimating its resistance, by evaluating the uniformity assessment and crack depth (if present) in the element.

Test procedure

A practical way of obtaining the depth of cracks found in pile caps, by means of ultrasonic waves, is through the discontinuity of the Time x Distance curve. When a pulse crosses a fissured region, the time it takes to go from the transmitter to receiver is larger, when compared to the time spent in a region with no cracks. When generating the mentioned chart (time x distance) a straight-line trend is expected for a uniform material, however, one may observe a discontinuity indicating the presence of cracks. The depth of the crack (h) is obtained using the following equation.

$$h = \frac{L}{2}\left(\frac{T_2}{T_1} - \frac{T_1}{T_2}\right) \quad \text{Eq. (1)}$$

2.3 Estimated expansion rate

The expansion rate represents the concrete swelling (in microstrain) in accordance with AAR. To calculate the mean expansion rate, a particular section is selected, and the sum of width of all cracks present on the surface of this particular area is divided by the total length of the cracks in the corresponding direction (horizontal or vertical).

The values obtained were:

- Mean vertical expansion rate: 40 $\mu\varepsilon$/year
- Mean horizontal expansion rate: 30 $\mu\varepsilon$/year

2.4 Concrete core extraction and compressive strength test

To determine the mechanical characteristics of the concrete as well as visualize the depth of the cracks, three cores were drilled from the pile caps. After the extraction, the cylinders were subjected to normal stress test to determine the compressive strength and modulus of elasticity. The achieved values are shown in Table 1.

3 MAIN REINFORCEMENT

The detailing of pile caps with 3 piles disposed in a polygonal arrangement often requires the flexural reinforcement to be positioned parallel to the polygon sides. The amount of reinforcement bars placed on top of each pair of adjacent piles can be estimated as it is shown in the explanation below:

- Angle of the load dispersed from the column to the piles:

$$tg\theta = \frac{d}{\frac{L\sqrt{3}}{3} - 0.3a_m} \quad \text{Eq. (2)}$$

- Considering a Strut-and-tie model, the stress resultant on the struts (D) and tie (T) are given by the following equations, for balance of force at the pile foundations node:

$$D = \frac{R_{est}}{sin\theta} \quad \text{Eq. (3)}$$

$$T = \frac{R_{est}}{tg\theta} \quad \text{Eq. (4)}$$

$$T = \frac{R_{est}}{d}\left(\frac{L\sqrt{3}}{3} - 0.3a_m\right) \quad \text{Eq. (5)}$$

Where:
D = stress resultant (compression) on the concrete.
T = stress resultant (tension) on the reinforcement bars.
Rest = Resultant of forces at the top of the pile cap (worst case scenario).

L = Resultant Distance between two consecutive piles.
a_m = smallest dimension of the column section.
d = distance between the top of the pile cap and the reinforcement bars.

Check for allowable compression resistance of the concrete on the truss region

By calculating the area of the strut by the column and by the piles, it is possible to obtain the stress resultant acting on these areas:

- The allowable compression resistance of the concrete on the truss regions are:

$$\sigma_{c,truss} = \frac{3R_{est}}{A_p sin^2\theta} \leq 1.75 f_{cd} \text{ (by the column)} \quad \text{Eq. (6)}$$

$$\sigma_{c,biela} = \frac{3R_{est}}{A_{pile} sin^2\theta} \leq 0.85 f_{cd} \text{ (by the pile)} \quad \text{Eq. (7)}$$

- Main reinforcement
  The main reinforcement area is calculated by:

$$A_{st} = \frac{T}{f_{yd}} \quad \text{Eq. (8)}$$

For the methodology of calculation presented above, it was assumed that the reinforcement bars were to be places on the top of each two consecutive piles, "linking" them on a triangular shape. Nevertheless, the bars can also be laid on the sides of each pile if the following corrections are made:

- The resultant acting on the struts should be decomposed on the top of the piles accordingly to the sides of the triangle that links them. That being done, the resultant (T') on the tie is calculated as follows:

$$T' = \frac{T\sqrt{3}}{3} \quad \text{Eq. (9)}$$

The reinforcement required, on each side of the new triangle, is given by:

$$A_{st} = \frac{T'}{f_{yd}} \quad \text{Eq. (10)}$$

Considering the concrete pile cap structure presented in item 2:

- Allowable Compression stress on the concrete: 26.5 MPa;
- Acting Compression load calculated (due to dead and column loads): 4.494 kN;
- Assuming that the reinforcement bars were placed parallel to the sides of the pile cap;

Ast = 11.91 cm² = 6 ϕ16mm/side

4 ALKALI AGGREGATE REACTION SIMULATION

A mathematical model was developed using the ANSYS software (Finite Elements Method), and, with an addition of a sequence of command lines written in APDL language (Ansys Parametric Design Language), it was made possible to simulate the expansive effects on the structure due to AAR, via thermal expansion coefficient, α, dependent on the analyzed direction. The results for the analysis were given by means of orthogonal stresses and accrued deformations.

The premises adopted to create the model were taken from the Moxotó Hydroelectric Power Plant studies [1]. Even though both structures are completely different, the expansion hypothesis and reaction pattern have similarities.

At the Moxotó Power Plant, the concrete expansion rate variation, in the course of time, was determined by a logarithmic regression of measurements made by an inverted pendulum. Figure 4 shows a chart that represents this vertical expansion rate variation. The equation used to calculate this rate was:

$$\varepsilon ov = 180 - 46 * \ln(\mathrm{t})$$ Eq. (11)

Where:
t = time in years

Equation 11 can be used only for t ≥ 3 years. For t less than 3, the chart was considered a linear increase up to 130 με. The relation between the horizontal and vertical expansion rate adopted was 0.5, based on Larive's work, which studied cylindrical and prismatic concrete specimens submitted to AAR [1]. Figure 5 shows the adopted curve of the vertical expansion rate over time.

In addition, the expansion rate may be calculated as a function of the confinement stress which the element is subjected, using the equation below:

$$\varepsilon = \varepsilon ov - K \text{ x} \log(\frac{S}{S_o})$$ Eq. (12)

Where:
ε = Expansion rate due to the AAR (orthogonal directions x, y, z);
εov = Expansion rate without confinement;
So = Stress value from below the point when expansion is constant;
K = Constant equal to the angular coefficient from the expansion rate for the confining stress chart;
S = Confinement stress (Sx, Sy, Sz).

Su represents the stress value from beyond the point when expansion becomes null. The relation between the expansion rate, which depends on the analyzed direction and the confinement stress, is graphically expressed in Figure 6. The confinement stress values adopted were So= 0.3 MPa and Su= 4.0 MPa.

Based on the theoretical expansion rate and the Moxotó power plant work, in the case of this concrete pile cap it was assumed that the expansion rate was constantly increasing up to the third year and it kept constant after that, resulting on 40 με in the vertical direction and 30 με for the horizontal direction. Also, as in Moxotó, the expansion rate was calculated as a function of the existing confinement stress, so the same values for So and Su were considered for these calculations.

5 MATHEMATICAL MODEL

This item presents the conception ideas used to create and develop the mathematical model.

Some aspects of the modeling related to geometry definition, finite elements characteristics, material properties, boundary condition and loads applied are shown as follows.

5.1 General criteria adopted for the mathematical model

The representativeness of the mathematical model was dependant on several criteria, of which the most important were: definition of finite elements, level of refinement of finite elements mesh and the choice of parameters for AAR simulation. Hereinafter, it is presented the numerical simulation procedures.

- Geometry definition of the model using AutoCAD;
- Definition of finite elements characteristics that are more appropriate to represent the structure;
- Development of the finite elements mesh;
- Definition of boundary conditions;
- Activation of gravity acceleration to apply dead load;
- Applying dead load;
- Activation of a macro routine for the AAR.

5.2 Elements, mesh and boundary condition

A solid element present in a model contains eight nodes. Each node has three degrees of freedom, regarding the translation along x, y and z axis.

The mathematical model was developed considering the real geometry of the structural elements, as in Figures 7 and 8. The element meshes were conceived to permit the adjacent node of each structural element to be coincident. The nodes in the lower portion of the piles had all degrees of freedom restricted (translations in x, y and z axis), simulating a fixed support.

5.3 Materials Characteristics

The attributes considered for the modeling process are in accordance with the specifications contained in the structural project, since, by the most recent characterization of the concrete test results, there hasn't been any loss of its resistance nor of its elasticity modulus.

The lower portion of the pile caps had its elasticity modulus increased, due to the presence of reinforcement bars.

5.4 Dead load

The dead weight relative to the piles, pile caps and columns are applied automatically by the ANSYS software. The program computes the dead load taking into consideration the geometry and materials chosen for each element. For reinforced concrete, the density value admitted is 2500 kg/m³. The other elements, which were not modeled, had their load applied as pressure at the top of the columns.

6 ANALYZED SECTIONS

The sides of the foundation block (faces 1 to 6), the upper and lower sides and two internal sections AA and BB (shown in figure 8), were selected to evaluate the stress results calculated by the software.

The results are in accordance with three different situations [MPa]:

- Year 0 (conclusion of civil works): stresses due to dead load, without AAR;
- Year 60 (2011): stresses due to dead load and AAR acting for 60 (sixty) years;
- Year 100 (2051): stresses due to dead load and AAR acting for 100 (one hundred) years.

6.1 Main stresses – Year 0

For the Year 0 analysis, the distribution of stress was as expected: tensioning stress concentrated around the top of the piles, resisted by the reinforcement, and on the mid and upper sections of the foundation blocks, the compressive stress was more diffused and closer to zero around its top. The dead and column loads are the only solicitations on the structure, causing the maximum tensile stress to be low and equal to 1.79 MPa. The main reinforcement is sufficient to resist these tensions.

6.2 Main stresses – Year 60 (Present)

After 60 years, in the presence of continuous AAR, the configuration of stresses inside the foundation block has changed significantly. There is a higher concentration of tensile stresses in the interior of the structure.

At the sides of the pile caps, the tensile stresses are higher than the limit of the material, and the design of the reinforcement for this region didn't consider the introduction of these tensions with time. So, as a consequence, a pattern of map-cracking starts to appear on the lateral surface of the concrete.

At the lower portion of the structure, the tensile stresses are even higher; however the main reinforcement is able of resisting the solicitation, without cracking.

6.3 Main stresses – Year 100 (Future)

After a hundred years of the AAR taking place inside the pile caps structures, considering a constant expansion rate, it is expected the tensile stresses to rise, resulting on the enlargement of the opening of the existing cracks and the occurrence of new ones.

In agreement to the assumptions made for 2051, in some specific points, at the lower portion of the foundation blocks, the main tensile stresses reach levels that overpass the allowable tensile stress supported by the reinforcement, representing structural collapse.

7 CRACKS REPAIR AND CONCRETE SURFACE PROTECTION

The studies carried out on the foundation structures pointed out the need of immediate repair. Even though the tensions are still under the allowable stress limits for which it was designed, the continuous exposure to moist will bring the internal tensions even higher. It is important to establish a waterproofing system to minimize the moist absorption and further deleterious expansion of the concrete.

The cracks found on the surface of the pile caps should be repaired and sealed, avoiding further absorption of humidity and corrosion of the reinforcement bars.

Repair sequence:

- Preparation of Surface;
- Application of the hydrofuge system;
- Recovery of the deteriorated concrete, with or without exposed reinforcement;
- Sealing of cracks on vertical surfaces;
- Sealing of cracks on horizontal surfaces;
- Sealing the sides of the pile caps using an acrylic flexible system.

8 CONTROL GROUP MONITORING

After the pile caps have been repaired, it is imperative that visual inspections and monitoring of crack development of a control group containing these elements are still carried out every six months. It is possible to monitor the cracks evolution by installing pins along its vertical and horizontal directions, as shown in Figure 9.

9 CONCLUSION

The visual inspection indicated the presence of cracks on the surface of the pile caps, with width varying between 0.4 mm and 5.00 mm. The results from the ultrasonic and concrete core normal stress tests show that the depths of cracks vary from 5 to 24 cm. Cracks on the piles were not observed.

As there is no actual data of the concrete expansion behavior before present, the expansion rate was considered uprising for the first three years, and constant after that.

In 60 years, the pile caps expansion rate was estimated at around 40 $\mu\varepsilon$/year vertically and 30 $\mu\varepsilon$/year horizontally.

The mathematical model showed that, in 2011, the internal tensile stresses in the pile caps are sufficient to cause cracking at the concrete lateral surface, but are bellow the allowable stress limit of the reinforcement at its bottom.

For the year of 2051, the internal stresses exceed the allowable stress limit of the reinforcement bars at the top of the piles.

For the present year, according to the assumed hypothesis, the internal stresses caused by the AAR are not yet structurally deleterious.

In order to avoid corrosion of the reinforcement bars and direct contact of moist with the concrete, monitoring the enlargement of the existing cracks and waterproofing all the pile caps is mandatory.

10 REFERENCES

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TABLE 1: Mechanical properties obtained from the axial load tests carried out on the concrete cores extracted from the pile caps.

Specimen	Location	fcm (MPa)	Ec (GPa)
CP1	P84	32.7	-
CP2	P81	34.3	29.1
CP3	P43	26.5	21.7

FIGURE 1: Edifício 10's pile cap.

FIGURE 2: Cracks found on pile cap.

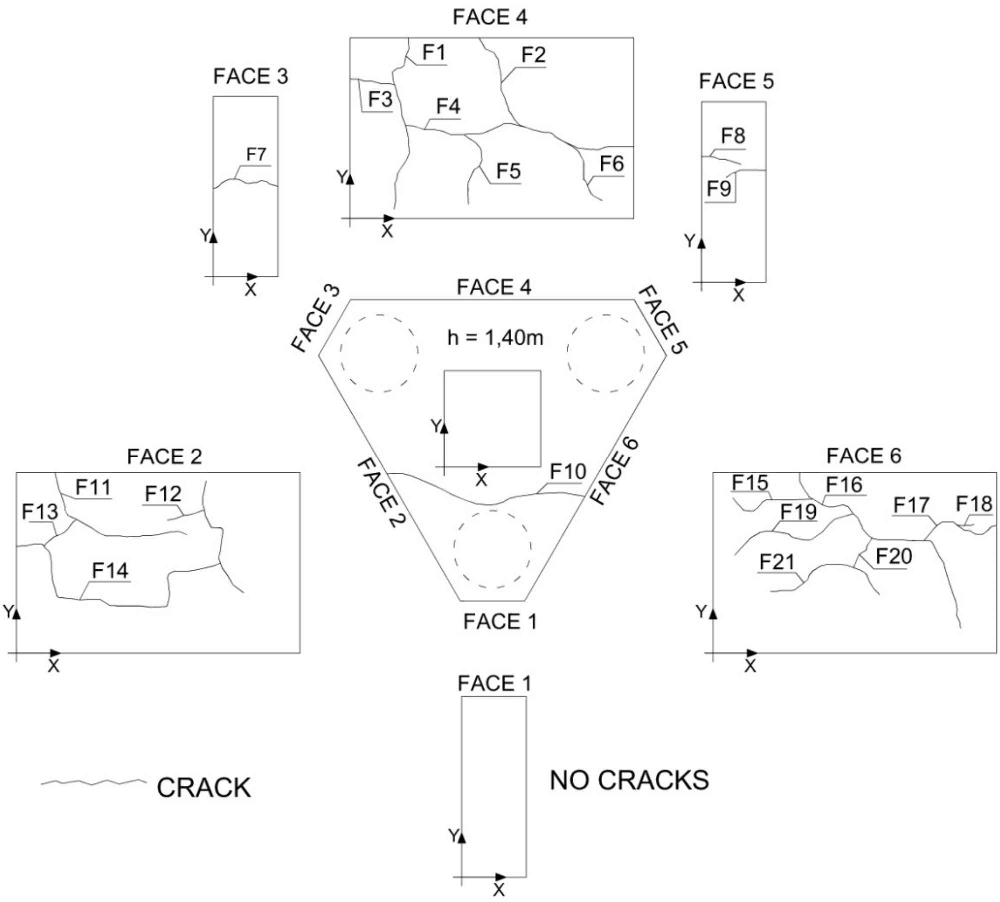

FIGURE 3: Cracks mapping of the pile caps.

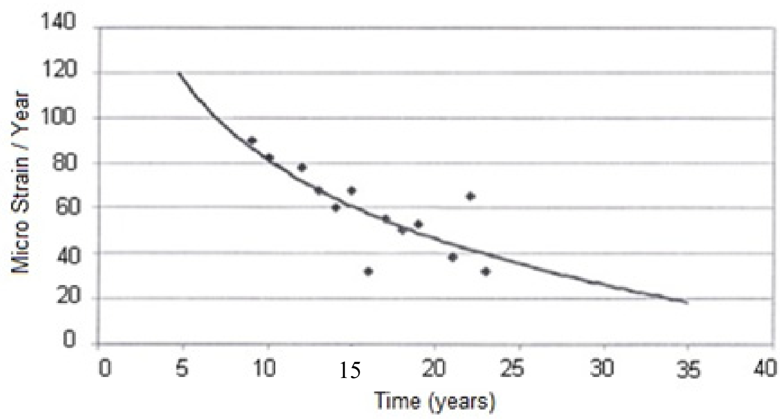

FIGURE 4: Concrete expansion with time.

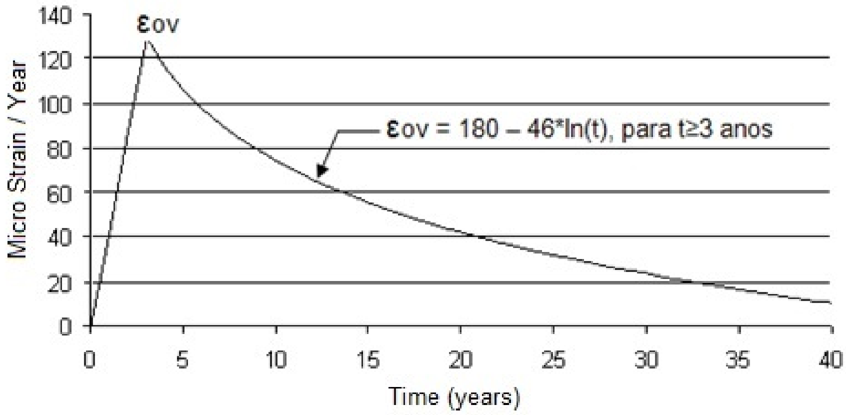

FIGURE 5: Expansion rates with time - Moxotó Hydroelectric Power Plant.

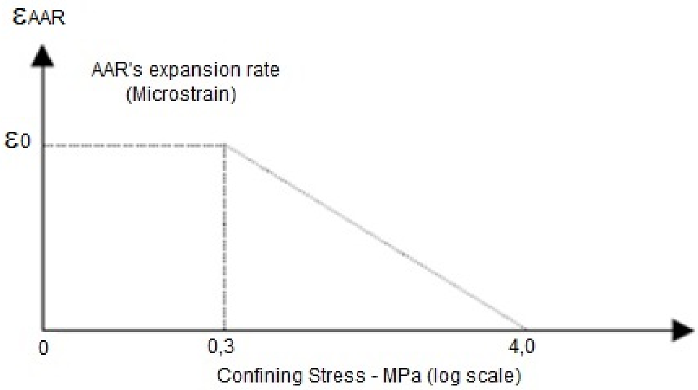

FIGURE 6: Relation between the expansion rate and the confinement stress (no specific direction).

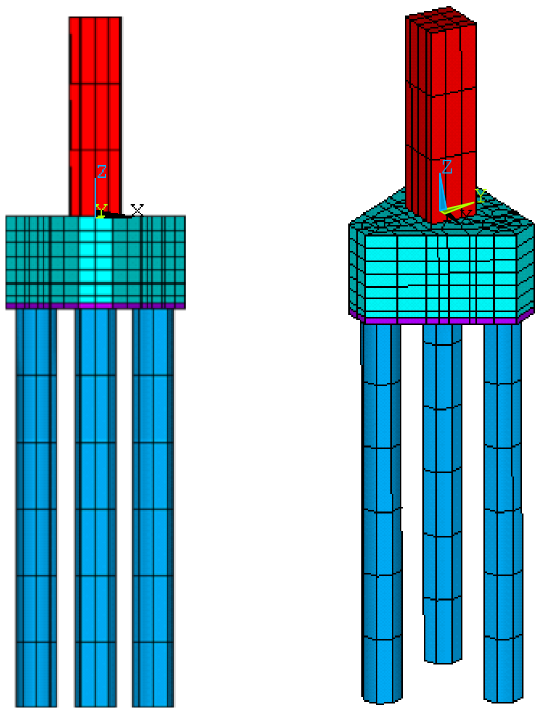

FIGURE 7: 3D view of the model.

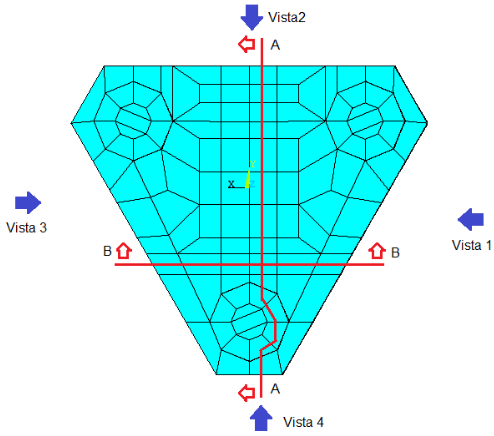

FIGURE 8 : Faces and sections of the pile caps studied.

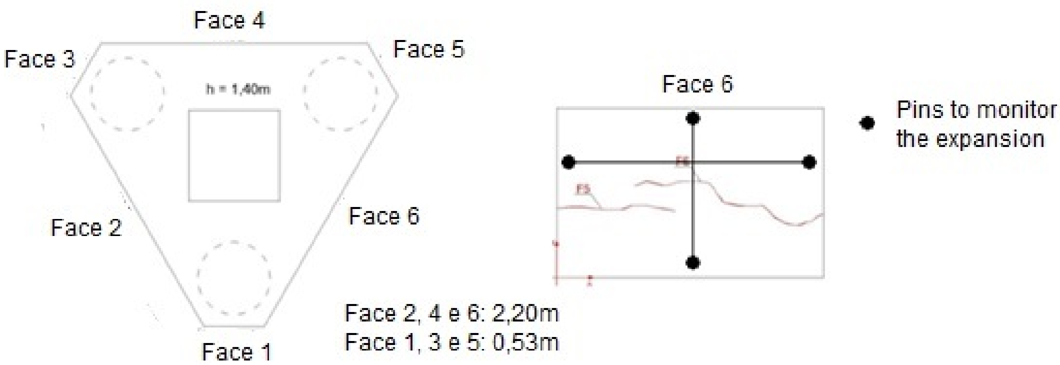

FIGURE 9 – Column 34, Face 6.