TOPIC 2 - Technology, techniques and training in Preservation and Conservation.
ANALYSIS OF STRUCTURAL AND GEOTECHNICAL BEHAVIOUR

Case Study: CONSOLIDATION OF THE BASEMENT FLOOR OF THE CATETE PALACE - Rio de Janeiro

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Brief historical background

The Catete Palace, formerly the residence of Antonio Clemente Pinto, Baron of Nova Friburgo and a wealthy coffee planter, was projected and built by Gustav Waehneldt, a German engineer. Work on the building was commenced in 1858 and continued until 1867, with renowned artists taking part in the construction. The edifice was sold in 1890 to the "Companhia Grande Hotel Internacional", and was transferred in 1896 to the Bank of the Republic of Brazil, becoming on that same year the property of the Federal Union for the purpose of housing the Government of the Republic, a function which it continued to serve until 1960. The premises were declared a National historical and artistic monument, under the terms of Process Nº 153-T and Nº 181-T, Historical Record Book, page 3, registered under nº 7, and Fine Arts Record Book, page 5, registered under nº 20, both entries dated April 6, 1938.

The main building exhibits neo-classical lines, three stories high and a square building plan. The walls are of stone masonry; with white and rose marble façades. Internally, linking the ground floor to the stately first floor, an imposing staircase enhanced with marble and bronze gives access to richly appointed rooms decorated with stucco, paintings, crystal chandeliers and superb furniture, that have witnesses the principal political events of Brazil's republican history. The premises comprise a very large 24 thousand square meter area embellished with a stream, bridges a fountain, grottoes, statues and large trees.

Construction of the Rio de Janeiro underground public transportation system, which started excavating section 23 of the project (comprising the Gloria-Catete-Largo do Machado districts) in January 1978, resulted in 30 mm contractions upon the front and 5 mm in the rear affecting the Palace at the time when the underground gallery was opened, the mass of terrain having been broken edgewise, with one principal crack and micro-plans in the region adjacent to the gallery, due to its proximity to the Catete Subway Station. Upon termination of the work on the gallery, in 1977, the speed of the pressure diminished and finally became stabilized sometime between 1978 and 1979. In 1983, and with the subway system fully in operation, new land movements were observed at an average velocity of 5 micra/day. An analysis of the cause of the reactivation of the contraction led us to the conclusion that cyclical micro sollicitations generated by the circulation of vehicles and trains were the cause of such movements.

Preliminary studies and structural instrumentation

Mention must be made of the fact that the gallery project comprised two basic aspects:

1º - That the underground transportation system would not create vibrations upon the buildings adjacent to the gallery;

2º - That maintenance of the equipment would follow the procedures established in the project.

Insofar as the first aspect, the type of permanent way used by Rio's subway was previously tested in France and considered to be the kind that presented the least vibration of the permanent way, when compared to others in Europe.

We proceeded, then, to observe the structural behaviour of the building in order to identify the causes and the kind of deformations being experienced by the structure. To do so, we devised an instrumentation plan for the building. We started by taking static tests, levelling the contraction pressure, using the alongameter, control of the opening of the fissures, and clinometry. We inspected all the networks of public utilities around the edifice and other installations in order to eliminate any possibility of hauling effects upon basement material. We also assessed the rainfall indices in the region during the last decade in order to relate such indices to any seeping of water into the area, but no abnormalities came to light.

We then proceeded to effectuate dynamic tests in order to assess the type and kind of vibrations transmitted to the supportin ground of the building's foundations. Measurements were taken of the ground and of the structure of the foundations, as well as of the structure of the building. The equipments that were used were: a seismograph with two geo-phones and six particulae accelerometers. Measurements were taken with several types of load, i.e., with surface vehicle transit stopped and underground transportation in motion, both accelerating and braking; with subway trains stopped and surface vehicle transit in motion, and with both kinds of transportation stopped. We obtained vibration frequencies in the range of 12 to 40 Hz, and the influence of surface transit proved to be in a lesser scale vis-a-vis the low speed that was observed.

The resulting measurements upon the structure of the foundations and upon the structure of the building did not present significant values. Such results served to confirm our opinion that the structure of the building considerably dampens the effect of vibrations and did not need reinforcing, and that the cause of the lesions was to be found in the deformities of the layers of the basement floor of the edifice.

Geological and geotechnical characteristics of the foundation grounds

The geological profile of the foundation grounds of the building can be considered to comprise a sandy matrix with an average thickness of 9,0 m and leaning from the rear towards the front and in the direction of the subway with a depth varying between 12,0 and 17,0 m. Under the sand layer there are thin discontinued poxkets of organic silted clay and sandy clay (also known as an erratic profile).

As we have already stated, during the period in shich the excavation of the gallery was being carried out, some contractions of the order of 30 mm were observed in the front part of the building, and of 5 mm in the rear area. Considering that the lateral diaphragm wall of the subway gallery is located at about 2,0 m distance from the foundations of the building, a number of analyses by finite elements were made, which showed that the vertical elastic deformations in the surface of the rear areaor backfill, could attain up to

15 mm. It is a well-known fact that the value of the deformation experienced by the rear wall was about 5mm, which falls within the possible deformation values imposed upon the back-fill, considering an elastic deformation regime ascribed to the mass, by the relief of tensions caused by the excavation of the gallery. The same assertion in not valid insofar as the displacements experienced by the front wall, i.e. 30mm that were observed and can be considered to be excessive for an elastic regime, caused by plastic components resulting from the deformation of ruptured surfaces in the zone of influence of the foundations, as observed in the building, One line of fissure parallel to the axis of the gallery sections the building in two, from the foundations all the way up to the roofing, and adjacent ground. We consider this to be the limit level between the plastic and elastic deformation.

Analysis of tensions and of the deformation of the sub-basement

We then proceeded to analyze the tensions and deformations in the sub-basement, considering that existing tensions added to the induced tensions due to the dynamic sollicitations that infinitely repeated load produces more and more a decrease in the value of the admissible tension of the geological layer. We made a parametric analysis in order to assess the increase of tensions in the field. Excessive movements caused by the excavation of the gallery imposed deformations that were predominantly of a plastic nature, generating ruptured micro-levels, and we concluded that the soil in the region of the foundations of the building must have had some residual resistance or perhaps low values in its module of elasticity. If this consideration is added to the decrease in the resistance capacity of the ground, because of the repetitive effect of the dynamic load, the mass, which seems to be stable, following the criteria established for a general analysis of the equilibrium-limit type, when the increase of tensions due to dynamic sollicitations is considered, it can be surmised that such an increase of tensions would be enought to reactivate ruptured micro-levels, thus causing new contraction pressures.

Cause of tensions and deformations in the sub-basement

Adter vibrations in the sub-basement were detected, we proceeded to research the cause of such vibrations. From the analysis of the results obtained through the geo-phones installed on the ground of the foundations of the building, we were able to observe that certain trains of the subway system presented irregularities in the wheels, exhibiting speed peaks that resulted in abnormal loads as related to the average speed observed.

A research project developed by the Subway System revealed that the corrugation of the railroad tracks was of small significance. But irregularities in the wheels were identified, characterized by the existencce of calluses which provokes serious efforts and strain upon the permanent track. Elimination of the calluses was recommended, and periodic maintenance with a turn-round lathe machine, which lad not been used for some time.

It is well-known that sand is the type of soil that becomes more easily compacted through vibration, and in order to achieve greater density in a short period of time, resonance frequency is utilized. It has been observed that the optimum interval for compressing sandy soils, insofar as vibration with compressing rolls, is between 0,5 and 1,5 times the natural frequency of the soil. Consequently, one should endeavour to use equipment that vibrates at frequencies between 25 Hz and 33 Hz.

The nearest that forced frequency can be to the natural frequency of the soil, the deeper the abatement of such soil shall be, through repetition of the load. Data obtained through the geo-phones installed in the premises enabled us to observe that forced frequencies are near to the level of the natural frequency of the soil that supports the foundations of the building, and this explains the permanent movements that were detected by the instruments.

It is also accepted that irregularities on the permanent track and on the wheels, due to defective maintenance, provoke high speeds with frequencies in the natural frequency as the sandy soil. Multiplication of such irregularities trhough the successive passing of trains in those conditions can produce significant tension pressures.

Taking into consideration the above mentioned fact and the need to stabilize the building for the purpose of restoring its highly valued artistic and aesthetic elements, the decision was made to consolidate the sub-basement geological layers.

Project of reinforcement and consolidation of the sub-basement

Consequently, a procedure was adopted to induce an improvement in the characteristics of resonance against sub-basement shearing and compression, so that dynamic sollicitations should not give cause to any movements in the affected area.

Therefore, consolidation of the terrain through compactation injections was recommended, using consolidating mortar and gel, considered to be the best solution from both the technical and economical points of view. Consolidation injections were prescribed, to be performed and attaining variable depths, between 6m and 9m, in accordance with the involvement of the mass that was to be treated.

The project estimated that the treatment would be carrid out in three stages:

- 1 st stage - Injections upon the foundations of the peripheral walls;
- 2 nd stage - Injections upon internal walls parallel to the façade;
- 3 rd stage - Complementation with injections upon the front and internal walls, except those already treated during the 2 nd stage.

  Closing and consolidation of existing fissures in the masonry of the edifice.

Rigorous <u>pari passu</u> inspection was foreseen through the use of instrumentation, of deformations suffered by the edifice during the works and in the latter accommodation phase. Special care had to be taken with the restored artistic elements in carrying out internal and external work, and for the recomposition of such items. Every care was taken, during the injections, to locate and avoid any interference with the nearby network of public utilities.

Specifications for consolidating the walls

In the case of fissures with openings of more than 15mm, holdfast clamps and injections of sand mortar and plain mortal were prescribed. For those with openings of more than 10 mm, injections of sand mortar and mortar were recommended. For fissures with openings of less than 10 mm, mortar and low density epoxy resins. were indicated.

Specifications for perforations and injections

1 - Perforation: effectuated with rotating drills, with the help of a suspension of bentonite, avoidance of closing drilled walls and allowing for the installation of injection tubes. During the works, the length, inclination and removal of obstructions from all orifices was checked to verify that they were in accordance with the project.

2 - Cement injections: The following pieces of equipment were used: mixer, sieves, shaker, injection pump, manometers, escape valve, and pluggers. The mortar consisted of cement, water, bentonite and additives.

3 - Gel injections: The option was made in favour os Single Flow or Pumping Injection of pre-mixed material. The following pieces of equipment were used: separate containers ofr the different components, regulator container equipped with constant level, separate for each liquid; mixer,injection pump, and pressure hose.
The recommended gel composition was: drink water, sodium silicate and ethyl acetate. Prior to systematically starting the injections, solidification tests were made to check the injectability of the soil layers of the subbasement, the efficiency of the treatment and optimization of the procedure.

4 - Topographic control: With the purpose of achieving rigorous control of the movements of the building while the drilling and injection operations were being carried out, a daily review of levels of the building was effectuated. This review was useful to indicate, while the work was in process, certain modifications in methodology, chiefly during the drilling phase. Two electric alarm systems were also installed, in order to instantaneously detect any excessive movements which could jeopardize the stability of the building, mainly during the injection phase.

5 - Assessment Tests:

5.1 - SPT Tests: We proceeded to make standard penetration tests which enabled us to compare the results before and after treatment. All profiles exhibited significant improvements.

5.2 - Pressure metric tests: Pressure metric tests were carried out to evaluate improvements in the characteristics of the grounds after treatment, and the results of such tests were positive.

5.3 - Seismic Tests: Comparative tests measuring the velocity of propagation of sound upon the grounds, with and without treatment, were carried our.

Conclusions

The building was kept under observation since November 1983. One year later, at a time in which we had already gathered sufficient data, the Consolidation Project was prepared and which was concluded in September 29, 1984.

The works started in January 15, 1985, and took six months, The "a posteriori" observation period took 12 months.

All different stages of the project were executed within the scheduled time, and no major changes were made to the initial project. The building is presently totally stabilized, and in the final phase of restoration of its artistic elements.

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THÈME 2 - Technologie, techniques et entraînement dans le domaine de la Préservation et de la Conservation.

ANALYSE DE LA CONDUITE STRUCTURELLE ET GÉO-TECHNIQUE

Cas d'Étude: CONSOLIDATION DU SOUS-SOL DU PALAIS CATETE - RIO DE JANEIRO

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Le Palais Catete, ancienne résidence de Antonio Clemente Pinto, baron de Nova Friburgo, richissime planteur de café, a été bâti par l'ingénieur allemand Gustav Waehneldt. Les travaux de constructions ont commencé en 1858, continuant jusqu'en 1867, ayant participé dans ces travaux des artisted renommés. En 1890 l'édifice a été vendu à la société "Companhia Grande Hotel Internacional", puis transféré en 1896 à la Banque de la République du Brésil; dans la même année, l'immeuble a été incorporé au patrimoine de l'Union Fédérale pour devenir le siège du Gouvernement de la République, fonction qui a été maintenue jusqu'en 1961. L'édifice a été déclaré monument national historique et artistique, en vertu des procédures Nº 153-T et Nº 181-T, Archives Historiques, page 3, enregistré sous le nº 7, et Archives de Beaux-Arts, page 5, enregistré sous le nº 20, tous deux enregistrements datés du 6 Avril 1938.

L'édifice principal présente des lignes néo-classiques, avec trois étages sur un plan d'édification carré. Les murs sont en maçonnage de pierre et façades recouvertes de marbre blanc et rose. Dans l'intérieur, le rez-de-chaussée est joint au premier étage par un grand escalier d'honneur enrichi avec des ouvrages en marbre et en bronze, donnant accés à des appartements richement garnis de stuc, peintures, chandeliers en crystal et meubles de grand prix, et qui ont été témoins des principaux évènements politiques de l'histoire républicaine du Brésil. La propriété comprend un grand parc de 24 mille mètres carrés, embelli d'un ruisseau, des ponts, une fontaine, grottes,statues et de beaux arbres anciens.

La construction du Métro de Rio de Janeiro, dont les travaux de percement des galeries souterraines ont commencé en Janvier 1978, secteur 23 du projet (comprenant les faubourgs de Gloria - Catete, et Largo do Machado) ont été la cause de contractions de 30 mm sur la partie frontale et de 5 mm sur la partie derrière, avec des effets sur le Palais à l'époque de la construction de la galérie, produisant une brisure oblique de la masse de terrain avec une fissure principale et des micro-plans dans la région adjacente à la galerie, en raison de la proximité avec la Station Catete du Métro. Quand les travaux de la galerie ont été terminés, en 1977, la vitesse de la pression diminua et est devenue stabilisée entre 1978 et 1979. En 1983, avec les trains en pleine opération, des nouveaux mouvements de terre ont été observés, présentant une moyenne de 5 micra/jour. L'analyse qui a été faite pour connaitre la cause de cette réactivation de la contraction nous a permis d'arriver à la conclusion qu'elle était due à des micro sollicitations cycliques produites par la circulation des véhicules et des trains.

Nous avons doc procédé à faire des observations sur la conduite structurelle de l'édifice afin de pouvoir identifier les causes et le type des déformations subies par la structure. Pour le faire, nous avons mis sur pied un plan d'instrumentation et nous avons commencé par des vérifications statiques en mesurant la pression des contractions au moyen de l'alongamètre, par le contrôle des ouvertures des fissures, et par la clinométrie. Nous avons réalisé l'inspection de tous les réseaux des services publics situés autour de l'édifice afin d'exclure toute possibilité de halage du matériel du sous-sol.

Nous avons alore procédé à faire des essais dynamiques afin d'évaluer le type et la classe des vibrations transmises au terrain de support des fondements de l'édifice. Nous avons fait des mensurations du terrain et de la structure des fondements, ainsi que de la structure de l'édifice.

Projet de renforcement et consolidation du sous-sol

En conséquence, nous avons adopté une procédure destinée à améliorer les caractéristiques de résistance au cizaillement et à la compressibilité du sol, de façon que les sollicitations dynamiques ne puissent donner lieu à aucun mouvement dans le secteur affecté.

Pous atteindre ed but nous avons suggéré la consolidation du terrain par le truchemendt'injections de compactation avec du mortier et de consolidation avec du gel, solution qui nous a semblée être la plus indiquées au point de vue technique ainsi qu'au point de vue économique. On a prévu des injections de consolidation sous la base des blocs des fondements, atteignant une profondeur variable entre 6 m et 9 m, conformément à l'implication de la masse a être traitée.