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Innovative methodology for advanced structural condition assessment of tunnels

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ABSTRACT: A methodology for holistic structural condition assessment of tunnels through high technological monitoring tools is proposed in this article. The methodology is divided into structural evaluation in real time through pre-installed equipment at the construction phase, and to post event structural evaluation through robotic inspection. The first part focuses on the static evaluation of the permanent tunnel lining after a strong earthquake in real time through fiber optic measurements. For this purpose, an improved type of fiber optic sensors is used, installed on inner and outer transversal reinforcing bars at eight points along the perimeter, allowing the recording of the time history of the elongation developed during the seismic action. A reformulation of the Parc-Ang criterion has been developed, allowing accurate automatic estimation of the degree of damage in each critical section according to the Moment-curvature diagram and the overall degree of indeterminacy of the structure. In the second part, the innovative model for crack analysis is described extensively, through which it is possible to calculate internal forces at the position of the structure where the crack is occurred, but also to estimate the possible locations of invisible cracks in the back face, based on the measurement of cracks' geometrical characteristics (widths, depths, angles and spacings) with sensors placed in robotic tools. Soil pressures and the internal forces N, V, M along the perimeter of the tunnel cross-section are calculated based on convergence measurements along 5 cords. The appropriateness of the methodology has been validated through the European Research Programs Tunneling, Monico, Robospect and Resist while the numerical results presented in this paper are obtained by application of robotic measurements in autonomous software packages implemented in JAVA programming language.

1 INTRODUCTION

Many railway, road and metro-line tunnels are presently under construction in seismic prone countries. The tunnels are often located under densely populated areas and require very high standards of safety. There is an increasing awareness of the sensitivity of these structures to seismic activity. Moreover, the damage to the tunnel structure is difficult to assess and a damaged tunnel that has survived the major earthquake might not have the capacity to survive consecutive seismic aftershocks. Such aftershocks take place within few hours of the earthquake and have been reported to have an intensity of up to 90% of the earthquake intensity. Even if a tunnel is not in an earthquake prone area, though, if not properly preserved through periodic maintenance and repair of manageable problems, the tunnel owner will eventually have to choose from two very undesirable options: shut down the tunnel for excessive repairs, accepting the resulting impact on the highway or transit system, or invest in very costly reconstruction, also with potential system

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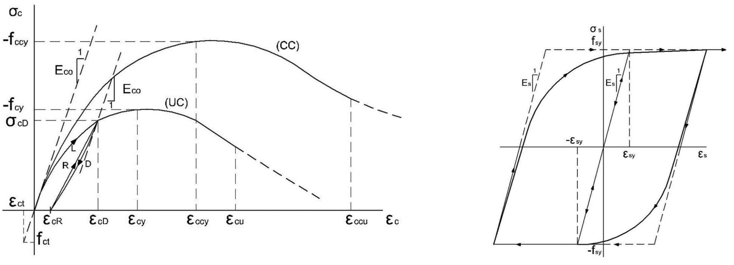

Figure 1. Constitutive law for concrete and steel.

repercussions during the period of construction. Tunnel linings are mostly constructed of reinforced concrete (r.c.). When subjected to significant ground shaking during an earthquake these members experience large deformations that force them into their non-linear range of behaviour. Even in the absence of an earthquake, though, tunnel linings undergo deformations under the effects of loads or changes in their constituent materials. The deformations of tunnel linings contain a lot of information about the lining state of health. By measuring these deformations it is possible to analyse the loading and ageing behavior of the structure and assess its safety.

Nowadays with the given technological development, it is now possible to monitor the project continuously throughout its life, but also to monitor the project periodically through inspections or after the observation of an event for the purpose of its structural evaluation. This monitoring can be done with multi-sensors that incorporate high-quality technologies such as for example fibre optics, laser scanners, etc.

In the first part of this work, a methodology is presented for the assessment of the structural condition of tunnels with technological equipment that is pre-installed in the during the construction phase and provides the ability of its continuous monitoring. Specifically a fibre optics based deformation monitoring system reports the deformation history of pre-selected reinforced concrete cross-sections of a tunnel lining during an earthquake. This history, with the help of a theory of seismic damage for reinforced concrete can provide an assessment of the section's damage. Subsequently, in the second part of the work, a methodology is presented for assessing the structural condition of tunnels without pre-installed equipment in the project, but from on-site measurements during the inspection procedure.

2 ASSESSMENT WITH PRE-INSTALLED SENSORS AT CONSTRUCTION STAGE

2.1 *Fibre- Optics. Methodology for structural assessement under seismic loading*

The most efficient procedure for estimating the resistance capacity of the structure through measurements of strains consists of the installation and recording of a strain gauges network along reinforcing steel bars in the inner and outer face of the lining, but its application is limited to new tunnel linings where the strain gauges can be installed in the erection phase so that the records represent the total state of the structural behaviour. The sensors are installed at the points of the tunnel cross section where maximum bending moments are expected to develop. The positions result from an analytical estimation. For the long term static loads the critical points are four and they lie on the vertical and the horizontal principal axes of the cross section. For the earthquake loads the critical points lie on two inclined axes of the cross section, so that finally the points of maximum bending moments are eight. At each critical point two sensors will be installed, fastened on the transversal reinforcing bars at the inner and the outer side of the thickness of the lining, so that the number of sensors required for each tunnel section will be 8x2=16.

Under the assumption of the linear distribution of the strains across the thickness and after taking into account the constitutive laws of the concrete and the steel, relating stresses and strains, for the loading and the unloading branches, are calculated the axial force N and the bending moment M, equal to the corresponding resultants of the stresses developed across the

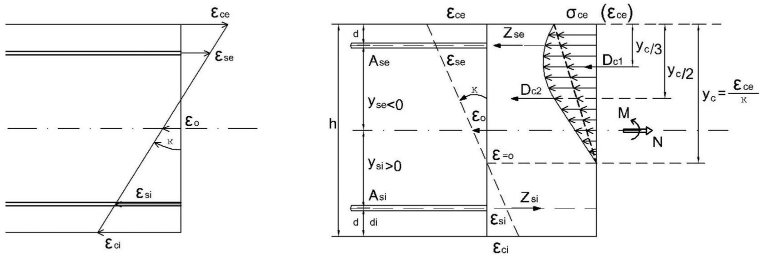

Figure 2. Cross section strains and internal forces.

thickness of the lining. The calculated values of the strains and the corresponding to them internal forces for the successive time instances during the earthquake motion, are referred to an axial force – axial strain diagram and to a bending-moment-curvature diagram. The last one, representing the development of the pair of values during the successive cycles of the seismic oscillation, constitutes the hysteresis loop pattern. According to the proposed methodology the characteristic points of all the semi- loops are calculated (points I,..,VII, Figure 3a), and finally the damage index D of the reinforced concrete member subjected to cyclic plastic flexural deformations, is estimated for the total number of the hysteresis semi- loops by using a revised expression of the Park and Ang damage criterion, resulting after the statistical elaboration of a big number of relative experimental tests.

2.2 *Calculation of cross section strains and internal forces*

From the records of the sensors are taken the strains ε_{si} and ε_{se} at the inner and the outer rebar respectively. Then the axial strain and the curvature of the reinforced concrete section are calculated from the relations: $\varepsilon_0 = (\varepsilon_{si} + \varepsilon_{se})/2$, $k = (\varepsilon_{si} - \varepsilon_{se})/(h - 2d)$, and by the assumption of linear deformation across the thickness, the strains at the inner and the outer surface of the concrete are also calculated. Detailed equations are introduced for the constitutive law of concrete and steel which are shown on Figure 1a & 1b, as follows:

$$\sigma_c = \begin{cases} \varepsilon_{cy}\lambda[E_{co}(1-\lambda)^2 + f_{cy}\lambda(3-2\lambda)/\varepsilon_{cy}] & (Loading - branch) \\ \sigma_{cD} + 8f_{cy}[[1 - 9E_{co}(\varepsilon_{cD} - \varepsilon_c)/4f_{cy}]^{0.5} - 1]/9 & (Deloading - branch) \\ pt\sigma_{cD}(\varepsilon_c - \varepsilon_{cR})/(\varepsilon_{cD} - \varepsilon_{cR}) & (Reloading - branch) \end{cases} \tag{1}$$

$$\sigma_s = \begin{cases} E_s\varepsilon_s & , |\varepsilon_{sn}| <= \varepsilon_{sy} \\ \frac{(\varepsilon_{s,n-2} - \varepsilon_{s,n-1})}{|\varepsilon_{s,n-2} - \varepsilon_{s,n-1}|}[-f_{sy} + E_s(\varepsilon_s - \varepsilon_{s,n-1}) - \frac{E_s(\varepsilon_s - \varepsilon_{s,n-1})^{\gamma}}{\gamma(\varepsilon_s - \varepsilon_{s,n-1})^{\gamma-1}}] & , \varepsilon_{sy} < |\varepsilon_{sn}| \end{cases} \tag{2}$$

Where: σ_c, σ_s are the concrete and steel stresses respectively, h is the the thickness of the concrete lining, d' is the cover of the reinforcing bars, fcy and fsy are the yielding stresses of concrete and steel respectively, ε_c is the concrete strain under compression, ε_{cy} the yield strain of concrete in compression, σ_c is the maximum stress of concrete, $\lambda = \varepsilon_c/\varepsilon_{cy}$, E_0 is the tangential modulus of elasticity of concrete in compression for $\sigma_c = 0$, Es is the elasticity modulus of steel, ε_{cD} is the starting strain of the deloading branch, σ_{cD} is the starting stress of the deloading branch, ε_{cR} is the starting strain of the reloading branch, n is the number of loading branch, ε_{sn} is the limit strain of the loading branch n and $\gamma = \frac{|\varepsilon_{s,n-2} - \varepsilon_{s,n-1}|}{|\varepsilon_{s,n-2} - \varepsilon_{s,n-1}| - 2\varepsilon_{sy}}$.

And the internal forces can be calculated as follows (Figure 2):

$$N = Z_{si} + Z_{se} + D_{c1} + D_{c2} \tag{3}$$

$$M = y_{si}Z_{si} + y_{se}Z_{se} + D_{c1}(\frac{h}{2} - \frac{y_c}{3}) + D_{c2}(\frac{h}{2} - \frac{y_c}{2}) \tag{4}$$

Where: A_{si}, A_{se} the area of the inner and external reinforcing rebars, σ_{si}, σ_{se} the stress at the inner and external reinforcing rebars, $Z_{si} = A_{si}\sigma_{si}$, $Z_{se} = A_{se}\sigma_{se}$, $D_{c1} = 0.5\sigma_{ce}y_c$, $D_{c2} = 0.67[\sigma_c(\varepsilon_{ce}/2) - \sigma_{ce}/2]y_c$, and $y_c = \frac{h}{2} - \frac{\varepsilon_0}{k}$.

2.3 *Modified Park- Ang criterion, the damage index calculation*

For calculating the section's damage index which represents the damage degree of the cross section after the earthquake, the total released energy quantity $E_{bf1} + E_{bf2}$ from the characteristic points of the cyclic semi-loops is estimated and divided by the initially available internal bound energy of the cross section E_u forming the ratio D. On each one semi-loops are defined the following seven characteristic points, denoted by the latin numbers I to IVII (Figure 3a), by which the moment-curvature trajectory is devided in regions corresponding to the different branches of the constitutive low of the concrete (reloading, normal loading, deloading and inactive branches). The hysteretic envelope consists of two branches, a right one for $k > 0, M > 0$ and a left one for $k < 0, M < 0$. Both branches are defined by two characteristic points, the yield point and the ultimate point. The total available internal bound energy equals the area included between the and the two branches of the hysteretic envelope (Figure 3d), which is:

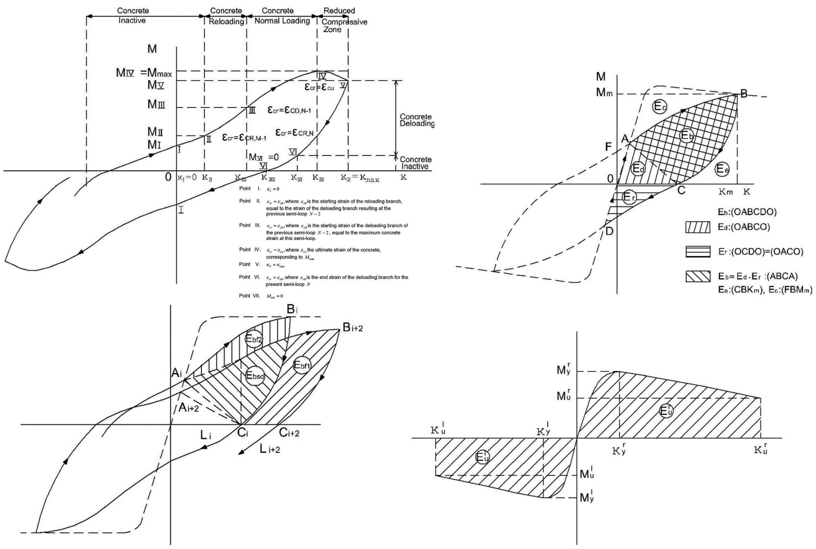

Figure 3. a) Characteristic points calculated on any semi-loop, b) On the diagram of any semi-loop (OABSDO) we calculate the complementary E_c energy, the released internal bound energy E_b, the elastic energy E_e, the dissipated energy E_d and the restoring energy E_r, c) For the numerator of the damage index the common area E_{bsd} between the iterative loops is taken account only once, d) The available internal bound energy E_u equals the area included between the $k - axis$ and the two branches of the hysteretic envelope and used in the denominator of the Damage index.

$$E_u = \frac{2}{3}(M_y^r k_y^r + M_y^l k_y^l) + \frac{1}{2}[(M_y^r + M_u^r)(k_u^r - k_y^r) + (M_y^l + M_u^l)(k_u^l - k_y^l)] \quad (5)$$

Where: $M_y^r, k_y^r, M_u^r, k_u^r$ are the Moment and curvature at the yielding and ultimate state respectively, at right side of the moment curvature diagram (Figure 3) and $M_y^l, k_y^l, M_u^l, k_u^l$ are the Moment and curvature at the yielding and ultimate state respectively, at left side of the moment curvature diagram (Figure 3).

Subsequently, the following relation can be expressed for the damage index of the cross section, in terms of the applied transversal force V at the free end of a canti-level and the corresponding transversal displacement δ of that point:

$$D = \frac{V_u \delta_{max} + \beta \int E}{V_u \delta_u} = \frac{E_{bf1} + E_{bf2}}{E_u} \tag{6}$$

The terms contained in the numerator and the denominator of the expression (6) at the right side of the above relation represent energy quantities. The physical reasoning of these quantities can be obtained by considering in detail the degradation procedure of the capacity for plastic deformation, as it is represented in any hysteresis loop pattern. The apparent effect of the cyclic loading is the gradual degradation of the stiffness between successive hysteresis loops, equal to the decrease of the slope of their loading branch. The gradual stiffness degradation is due to the permanent plastic deformation of the tensile rebars, accumulated during the repeated cyclic loading, resulting to increased crack widths and less contribution of the concrete to the resistance. Two kinds of stiffness degradation are distinguished:

The first order stiffness degradation concerns the difference of slope between two successive semi-loops of different maximum deformation, which results to be proportional to the difference of the areas of the two semi-loops. The sum of the differences of areas, taken along the δ -axis for the totality of the hysteresis loops of the pattern, equals the first term at the numerator of the damage index expression and constitutes the first order released internal bound energy E_{bf1}. The second order stiffness degradation concerns the difference of slope between two successive semi-loops of the same maximum deformation, which results to be proportional to the difference of the areas of the two semi-loops in the direction of the M- axis. The sum of these differences of areas for the totality of the hysteresis loops of the pattern, equals the second term at the numerator of the damage index expression and constitutes the second order released internal bound energy E_{bf2}. The term in the denominator of the damage index expression represents the initially available internal bound energy and equals the area of the total hysteretic envelope E_u. The energy quantities expressed in terms of transversal forces and displacements concerns the total elastic and plastic energy appearing along the total length of the structural member. In case where the estimation of the damage index in the region of a plastic hinge is of interest, at points where maximum bending moment is developed with zero shear force, are taken the same energy quantities per unit length of the plastic hinge region, expressed by the product M_k of the bending moment by the curvature. The released flexural internal bound energy E_{bfm} of the *m–th* semi-loop, equals the sum of the areas of the non common parts $E_{bf1m}E_{bf2m}$ of the released internal bound energy areas $E_{b,m-2}E_{b,m}$ of two successive semi-loops of order m-2 m(Figure 3c). Their common part is taken into account at the treatment of the previous semi-loops. It is $E_{bfm} = E_{bf1m} + E_{bf2m}$, where $E_{bf1m} = M_{um}(k_{um} - k_{u,m-2})$ for $|k_{u,m-2}| < |k_{um}|$, $E_{bf1m} = 0$ for $|k_{um}| < |k_{u,m-2}|$ and $E_{bf2m} = E_{b,m-2} - E_{bm} + E_{bf1m}$.

3 POST-EVENT ASSESSMENT WITHOUT PRE- INSTALLED SENSORS

The post event Assessment Tool is aiming to the preparation of a software for a near real time evaluation of the entities characterizing the structural condition of existing cast in situ or segmental reinforced concrete tunnel linings which is based on data received from measurements of the characteristics of cracks developed on the visible internal face and the deflections of points on it due to its deformation (Figure 4a). The main target of this part is the structural condition assessment of existing and not of new tunnels. In this case, the installation of a strain gauges network is not applicable. The assessment is based on measurements of visible consequences of the structural behaviour such as the crack patterns, the gaps at the joints between precast segments and the deflections developed at the points of the internal face of the lining. The increased accuracy of the modern equipment offered for that purpose reinforces the attempt for the establishment of an efficient detection system. A decisive part of this system is the preparation of the appropriate software module permitting the final structural condition assessment to be obtained. The initial data concerning the geometry, the reinforcement and the grades of the materials for the tunnel cross sections need to be listed in detail. The constitutive laws of the materials consisting of the stress-strain relations for the reinforcing steel, the concrete and the bond-slip relation between the concrete and steel are represented by appropriate analytical expressions. The time effects on the strength, the creep and the shrinkage of the concrete, are likewise represented by analytical expressions specified by the codes.

Through the Local Condition Assessment Module, the strains, the stresses, the internal forces, the available strength and the safety factors are calculated at the points of the tunnel cross section where cracks have been detected and measured.

Through the Global Condition Assessment Module, the same entities characterizing the structural behaviour for the totality of the points on the cross section's perimeter are calculated by using the data from the measurements of the deflections at eight points.

Additionally, the actual values of the water and soil pressures applied are calculated and possible cracks expected to have developed on the external invisible face of the lining are detected.

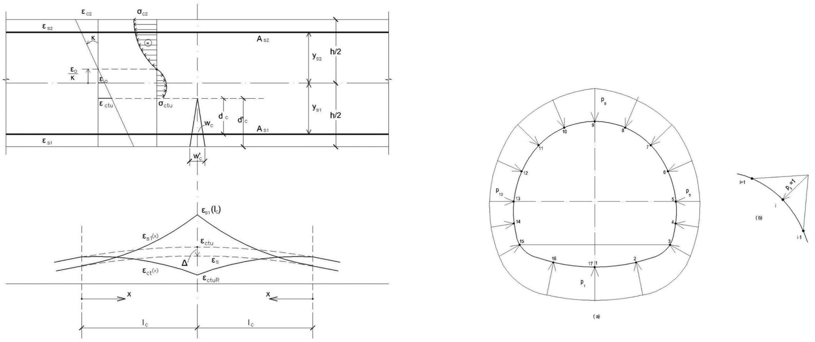

Figure 4. a) Cracking strains and geometrical characteristics of cracks (w- crack width and d- crack depth) b) Soil and water pressures on the model nodes.

3.1 *Local condition assessment*

The Local Condition Assessment Module, based on the measured depth, width and spacing of a group of horizontal flexural cracks observed at a position calculates the developed internal forces, the stiffness, the available strength and the safety factor at this position.

Under the assumption of the linear distribution of the strains along the thickness, the normal strains at any point of the cross section are calculated. The strains due to shrinkage are considered at cross sections where $maxM(V = 0)$ for external loads. At these cross sections the slip between concrete and steel is $\delta_b = 0$. Moreover, small values of concrete strains due to shrinkage at Stage I are expected to be developed ($|\varepsilon_{ci,s}| < |\varepsilon_{cs}|$), permitting that the assumption for the linear distribution of the strains is additionally extended to the concrete stresses with constant value of the elasticity modulus $E_c(ci, s) = E_{c0}$. Regarding the cracking strains, instead of the measured characteristics of a crack, the depth d'_c and the width w'_c, their effective values referred at the level of the tensile reinforcement $d_c = d'_c + y_{s1} - h/2, w_c = (d_c/d\prime_c)w'_c$ are considered (Figure 4a).

3.2 *Stage II conditions, operating state*

From known values of the cross section strains ε_0 and the material strains $\varepsilon_{s1}, \varepsilon_{s2}$ and ε_{c2} are calculated. The normal stress resultants for the reinforcing steel are then: $N_{s1} = E_s A_{s1} \varepsilon_{s1}$, $N_{s2} = E_s A_{s2} \varepsilon_{s2}$. And for the concrete, the axial force and bending moment can be defined as follows:

$$N_c = \frac{b}{k}[E_{c0}\varepsilon_{cy}^2[(1 + \frac{\varepsilon_{c2}}{\varepsilon_{cyc}})e^{\frac{-\varepsilon_{c2}}{\varepsilon_{cyc}}} - 1] - E_{c0}\varepsilon_{cty}^2[(1 + \frac{\varepsilon_{ctu}}{\varepsilon_{cty}})e^{\frac{-\varepsilon_{ctu}}{\varepsilon_{cty}}} - 1]] \tag{7}$$

$$M_c = \frac{b}{k^2}[E_{c0}\varepsilon_{cy}^3[[1 + (1 + \frac{\varepsilon_{c2}}{\varepsilon_{cyc}})^2]e^{\frac{-\varepsilon_{c2}}{\varepsilon_{cyc}}} - 2] - E_{c0}\varepsilon_{cty}^3[[1 + (1 + \frac{\varepsilon_{ctu}}{\varepsilon_{cty}})^2]e^{\frac{-\varepsilon_{ctu}}{\varepsilon_{cty}}} - 2]] \tag{8}$$

The total cross section resultants, the axial force *Nsk* and the bending moment *Msk* are:

$$N_{sk} = N_{s1} + N_{s2} + N_c \tag{9}$$

$$M_{sk} = y_{s1}N_{s1} + y_{s2}N_{s2} + M_c - \frac{\varepsilon_0}{k}N_c \tag{10}$$

The eccentricity of the axial force *Nsk* with respect to the member axis is: $e = \frac{M_{sk}}{N_{sk}}$ and the stage II Stiffness of the cross section is: $(EA)_{II} = \frac{N_{sk}}{\varepsilon_0}, (EJ)_{II} = \frac{M_{sk}}{k}$.

3.3 *Stage II conditions, ultimate limit state*

The ultimate limit state occurs for $\varepsilon_{c2} = \varepsilon_c$ and $\sigma_{s1} = -\sigma_{s2} = \sigma_{sy}$, assuming that the hardening of the steel as well as the tensile stresses of the concrete can be omitted as negligible. Subsequently the cross section resultants N_{rd}, M_{rd} are then calculated and the total safety factor of the cross section can be derived as the following ratio:

$$\gamma = \frac{N_{rd}}{N_{sk}} = \frac{M_{rd}}{M_{sk}} \tag{11}$$

3.4 *Global condition assessment*

The Global Condition Assessment Module, based on the measured deformations at eight points on the internal face of a tunnel cross section and calculates the same entities as those in the Local Condition Assessment Module and additionally the external soil and water pressures applied as well as the characteristics of possible invisible cracks expected to be developed on the external face of the lining in contact with the surrounding rock mass.

The global analysis of the lining is performed on a structural model as that shown in Figure 4 On the internal face of the tunnel cross section are defined 16 points, 4 of them at the intersection of the internal perimeter with the two principal axes of the cross section and the rest dividing the resulting quarters at quasi equal intervals. The reference line of the model is the polygon sequence connecting the 16 points, which are considered as the notes of the model. For segmental linings is suggested the points to coincide with the position of horizontal joints between the precast segments. The structure or the lining is represented by a sequence of 16 beam finite elements, each one between two successive nodes. The end points of the axes of the beam elements are put through rigid offset connections at a distance equal to h/2 from the corresponding end nodes in the direction of the bisectrix of the angle between the sides of the polygonal jointed to the node, where h is the thickness of the lining at that point.

A structural analysis software is used exclusively for the previously described structural model and the analysis is performed for the following load cases: 1) Shrinkage ε_{cs} resulting to node displacements $\delta_{sxi}, \delta_{syi}$ and the internal forces N_{si}, V_{si}, M_{si}, 2) the self weight g resulting to the nodes displacements $\delta_{gxi}, \delta_{gyi}$, and the internal forces N_{gi}, V_{gi}, M_{gi}, 3) the external soil and water pressures, in the radial direction of the lining and continuously distributed along it, defined by the values p_i at the nodes. The values of these pressures are unknown as they are increasing with time simultaneously with the deterioration procedure of the surrounding temporary supporting lining which is directly exposed to the corrosive environment of the rock-mass. Their actual values are calculated from the following analysis procedure: 15 separate analysis of the model are performed, each one for a triangular pressure applied on the beam elements k – 1, k and k, k + 1 with a unit pick value p_k = 1 on the intermediate load k resulting to the nodes' displacements *xi,k*, *yi,k* for the totality of the loads. (Figure 4b). If q_k are the actual values of the external pressures on the nodes, the superposition of the displacements for the three load cases have to satisfy the conditions:

$$\delta_{cxi} + \delta_{gxi} + \sum_{k=1}^{16} q_k \delta_{xi,k} = \delta_{xi} \tag{12}$$

$$\delta_{cyi} + \delta_{gyi} + \sum_{k=1}^{16} q_k \delta_{yi,k} = \delta_{yi} \tag{13}$$

with i = 1,3,5, ,15 the 8 odd nodes and xi, yi the measured relative displacements of these nodes with respect to the fixed node 1. From the solution of the system of the 8 + 8 = 16 linear equations result the values of the unknown external pressures q_k. The internal forces for the total pressure are:

$$N_{qi} = \sum_{k=1}^{16} q_k N_{i,k},\ V_{qi} = \sum_{k=1}^{16} q_k V_{i,k},\ M_{qi} = \sum_{k=1}^{16} q_k M_{i,k} \tag{14}$$

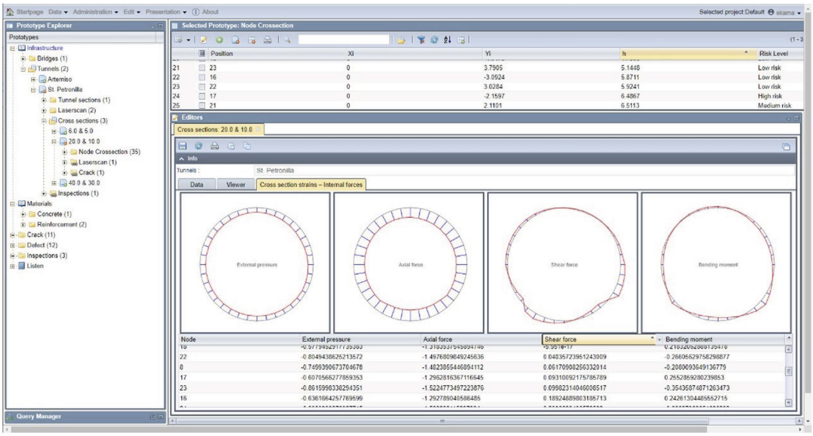

Figure 5. Software application in global condition assessment for pilot- test measurements at Petronilla Tunnel in Torino. The external pressures, the Axial Force, the shear force and the bending moment are calculated for measured tunnel section deformations and measured geometrical crack characteristics through robotic instruments.

Finally the internal forces for the superposition of the load cases are calculated as below:[1–4]

$$N_{sk,i} = N_{si} + N_{gi} + N_{qi} \tag{15}$$

$$V_{sk,i} = V_{si} + V_{gi} + V_{qi} \tag{16}$$

$$M_{sk,i} = M_{si} + M_{gi} + M_{qi} \tag{17}$$

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