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SESSION 3

Interpretative Models and Decision Making

Modèles d'interprétation et prise de décision

Erklärungs- und Entscheidigungsmodelle

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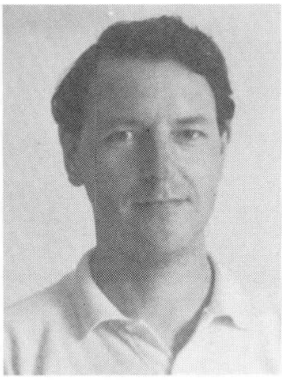
Dynamic Behaviour and Monitoring of Ancient Monuments

Comportement dynamique et surveillance des monuments

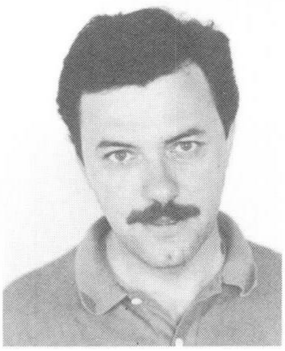
Dynamisches Verhalten und Überwachung von Denkmälern

Carlo BLASI
Dipartimento di
Ingegneria Civile
University of Florence, Italy



Born in 1948, got his architectural degree at University of Florence. Researcher in the field of consolidation of historic buildings.

Paolo SPINELLI
Dipartimento di
Ingegneria Civile
University of Florence, Italy



Born in 1950, got his civil engineering degree at University of Florence. Associate Professor, works in the field of theory of structures and buildings.

Andrea VIGNOLI
Dipartimento di
Ingegneria Civile

Born in 1950 got his civil engineering degree at University of Florence. Associate Professor, works in the field of earthquake engineering.

SUMMARY
Some experiences are illustrated for the dynamic identification problem in the case of ancient monuments. The dynamic testing techniques can provide a helpful control system for the survey and monitoring of historic structures.

RESUME
On présente des expériences pour la connaissance du comportement dynamique des monuments. Les tests dynamiques peuvent constituer une technique utile pour le contrôle et la surveillance des structures des constructions historiques.

ZUSAMMENFASSUNG
Wir stellen die Erfahrungen bei dynamischen Versuchen an Denkmälern vor. Dynamische Tests können eine nützliche Technik zur Kontrolle und Überwachung der Strukturen von Historischen Bauten bilden.

1. INTRODUCTION

The development of dynamic computation techinques for the study of the dynamic behaviour of structure, together with more efficient prediction of the behaviour of structure under dynamic loads (seismic events, wind etc.) also produced a modification in the in situ techniques of structures' testing.

In fact with the comparison of the dynamic results obtained with real experiments (on the physical model) it is possible to obtain information about the structure's statical situation. With the variation of the parameters which influence the dynamic response of the mathematical model it's possible the "tuning" of the model in order to obtain as outputs the same dynamic response parameters of the actual physical one. This is the sense of the word dynamic survey as it's possible to have a structural survey of a monument through dynamic testing.

This is important expecially for those type of structures (as monumental buildings) for which statical tests are impossible or not significant and so dynamic testing techniques give important documents for the "surveying" of the monument. The dynamic survey can be put so beneath the geometric and architectural ones to furnish important elements for the exact understanding of the structural state.

This survey is useful also to give elements to judge about the safety level of the monument, for the control and monitoring of it. In fact the repetition of the dynamic survey after some events which can effect the positively (as consolidation and repairing works) or negatively (as seismic events or wind storms) can give an important identification document for the monitoring of the monumental building.

In these last years the dynamic tests in the structural field have shown important development and refinements.

Particularly they have been devoted to completely define together with eigenmodes, eigenfrequency and relative damping factors also the transfer function (or, better, the imaginary best of this function, calculated in Laplace domain) also called frequency response function.

Of course this function and all the above mentioned dynamic properties are uniquely defined for linear structures whilst for non linear structures the above mentioned properties have sense only in the neighbourhood of a certain equilibrium position.

Now usually as the monumental buildings are masonry or stone structures, the non linearity is caused by the fact that the material does non resist to tension forces and the supports are usually monolateral ones.

But of course the above mentioned non linearity shows itself only in case of large movements and high level excitations.

However the experiences in the field of dynamic tests of monuments (see references /1/-/9/) have clearly shown that it is not usually possible for this type of structures to induce high level vibrations on one hand because the structural masses are very big and on the other hand because these vibrations can be dangerous for secondary or decorative elements of the building.

For this reason it is usually possible also for stone or masonry structures, to use a linear elastic behaviour model for the interpretation of dynamic tests. So the dynamic survey through the recording of the above mentioned dynamic behaviour quantities have sense also for stone and masonry building.

Neverthless, it is sometimes preferable for slender structural such as columns or columnades, to develop a non linear analysis for the control of tests' results. In ref. 10 a method for the non linear analysis of stone blocks columns is developed in the following applied for columns and columnades in Roman Forum.

The dynamic excitation techniques of the structures can be grouped in two great families that is the natural family and the artificial one. The first is of course the easier for the operator as it is not necessary to supply and study a specific excitation technique. To this group belong wind excitatio, seismic excitation, traffic, but of course to furnish useful indications they must themselves be identified in order to permit an exact evaluation of the transfer function.

Among the artificial methods when a particolar vibration mode is to be excited, the technique of the imposed deformed shape suddenly released can be usefully employed.

The other artificial methods are based or upon the use of vibrators which can be fixed to the structure and impose forced vibrations, or on the use of vibrating tables which give impressed displacements to the footings. Of course for the monumental buildings only the first type of technique can be used. The most used instrument is certainly the mechanical vibrodine which cause harmonic excitations, variable in frequency; the electrohydraulic shakers are used to give excitations variable in time with any desired law (i.e. stochastic or not sinusoidal).

Again (ref. /11/-/25/) gas excitators and electrohydraulic ones have been used to impose impulse forces to excitate a particular eigenmode linked to the application point.

At last another type of excitation is the microseismic excitation caused by artificial explosions underground, but of course this type of test can be only rarely applied in historic centers and for monumental building.

2. EXPERIENCES

The dynamic surveying techniques have been employed for many years by the Department of Civil Engineering, sometimes in collaboration with C.R.I.S.-E.N.E.L. (and ISMES), to study some monuments in Florence and in Rome.

2.1. Columns and columnades in the Forum in Rome

In the occasion of the consolidation works of some columnades of the Roman Forum, dynamic testing has been achieved to study the structural behaviour and to determine the more appropriate consolidation works necessary for seismic protection.

Besides a computation non linear method has been developed both to obtain the "tuning" of the dynamic parameters of the tests with those of the computations, and to study and predict the responses under seismic important events which cause sections partialization with high level displacements.

Particularly the following monuments have been tested:

Castore and Polluce Temple, Vespasian Temple, Saturn Temple, Foca's Column.

The tests have been conducted exciting the monuments with impulsive load tests, and with a small vibrodine. The tests for the Foca's Columns have been repeated also after the consolidation work, "monitoring" in this way the monument. A variation in natural frequency from 0,8 Hz to 1,73 Hz has been recorded. For Castore and Polluce Temple also a finite element mathematical model has been applied.

The results have allowed to identify the most damaged structures and to turn a numerical non linear model for the study of the seismic response of monumental buildings.

In figures 1 and 2 some results of the tests are illustrated.

In figure 3 the graphs of the displacements (a) and of the rotations (b) are reported, as obtained on the numerical model with a sinusoidal motion at the base. In figure 4 two examples are reported of the experimental results in term of displacements of the top of the column of Foca before and after the consolidation works.

2.2. Colosseum

For a sector of the boundary wall of the Colosseum in Rome some dynamic tests before and after consolidation works which tends to anchor the external wall (which shows a dangerous out of vertical alignment) to the radial ones.

The tests have been conducted through impulse excitations and have been the oscillation modes of the examined wall recorded.

The results has allowed to evaluate the effectiveness of the upgrading works done, both as improvement of the anchorages between the masonry walls, and as global stiffness.

In fig. 6 the computation numerical model using finite element method of the examined sector of the monument is illustrated. With this model the tests have been interpreted. The frequency of the first mode is increased from 1.3 Hz to 1.8 Hz, as a consequence of the consolidation works.

2.3. Chains of S. Maria degli Angeli Dome in Rome

A series of tests to determine the natural frequencies of the 84 chains of the Michelangelo cloister of St. Maria degli Angeli church in Rome, has allowed to determine the existing tension in the chains and to evaluate the safety existing limits.

The tests have been repeated on some chains, after the works conducted at the upper floor in the museum and successive future controls and monitoring will be achieved during the work of positioning of further chain necessary to diminuish the chain tensions.

In figures 7 and 8 the photos of part of the chain system is illustrated, together with the deformometers applied on the chains for the dynamic tests.

2.4. Chains at St. Maria del Fiore cathedral in Florence

An investigation similar to that reported at the above paragraph has been develped on the whole chain system of the Cathedral of S. Maria del Fiore in Florence. With the tests it has been possible to evaluate the tension state of the chains and to start a study on the static behaviour of the entire building.
Besides, for a continuos in time control of the stresses, some electric extensometer which are being controlled systematically.

2.5. Claudio's aqueduct in Rome

Some parts still standing of the Claudio's aqueduct in Rome are in the southern suburbs of rome, just in adiacence to the railway.

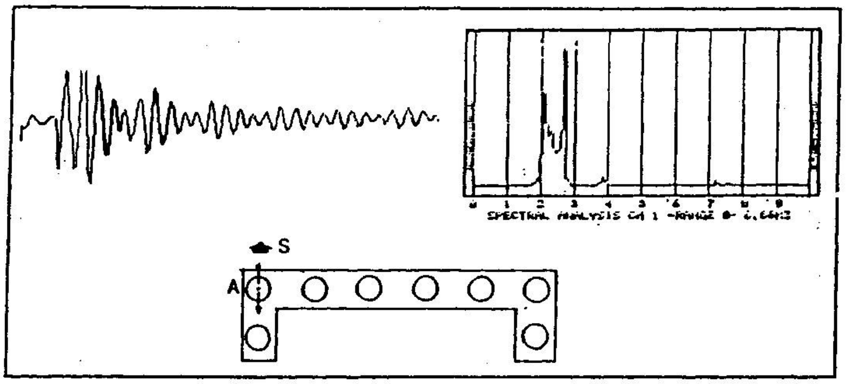
In the course of the consolidation works recently done, some dynamic tests have been developed for a survey of the structural situation of the monument and for the control of the effects of train passage, also in view of possible increase in railway traffic.

2.6. Tests on florentine monuments, in collaboration with CRIS-ENEL

In collaboration with C.R.I.S.-E.N.E.L., under the protection of Commune of Florence and the Superintendent of Monuments, some dynamic tests have been conducted and other are in programm on some monuments in Florence. In particular Brunelleschi's dome and Arnolfo's Tower have been tested. The excitation was given by natural causes (wind) for the Brunelleschi's dome, whilst for Arnolfo's Tower besides the wind action, also a vibrodine at the top of the tower was employed for excitating the structure.

The tests have been conducted by some technicals of ISMES.

The speed of some points of the structure was recorded with a laser type technology. However more details on this tests and those on Temple of Marte Ultore in Rome will be presented in the works of Chiarugi, Castodi, Giuseppetti, Fanelli, Petrini presented at this Colloquium.



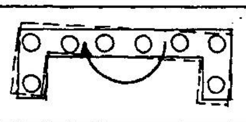
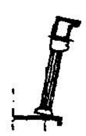
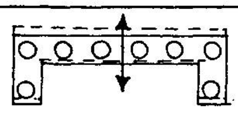
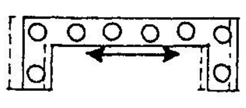
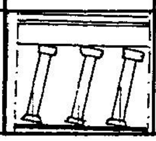
	Experimental study	Hz	Numerical models	K	Hz
TEMPLE OF SATURN		1.50		600 kg/cm^3	1.87
		1.85			
		2.10			2.05

fig. 1 Tempio di Saturno: Displacements concerning 3 eigenmodes

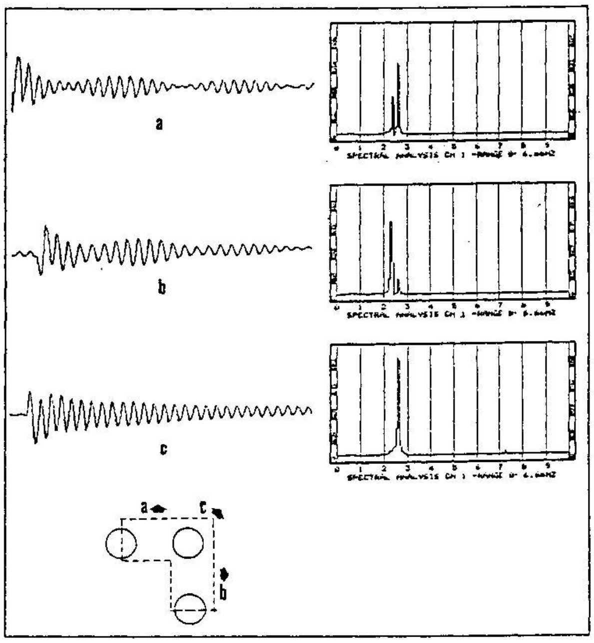
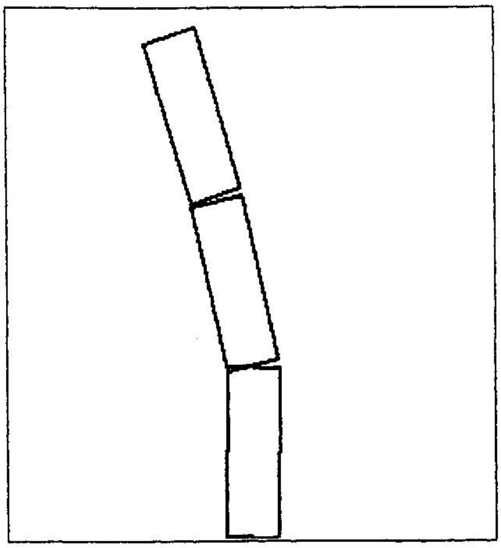


fig. 2 Tempio di Vespasiano: Displacements concerning 3 eigenmodes



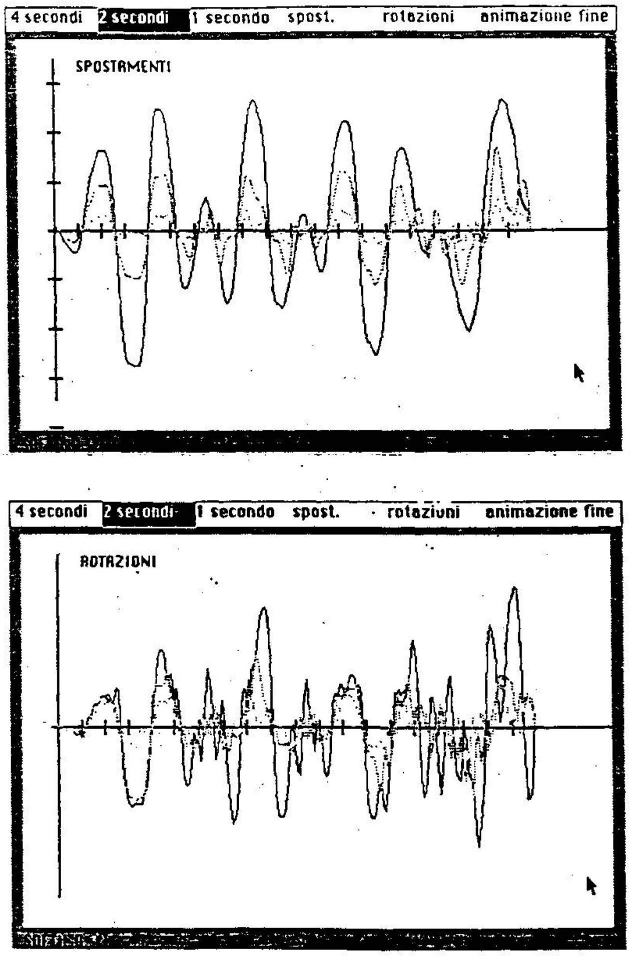


fig. 3 Colonna di Foca: Numerical non linear model and results (displacements and rotations)

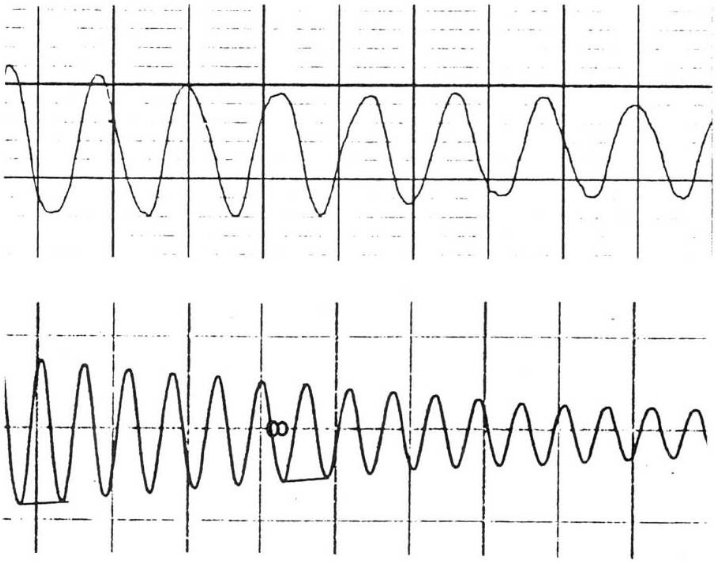


fig. 4 Colonna di Foca: Some examples of displacements effected before and after the consolidation

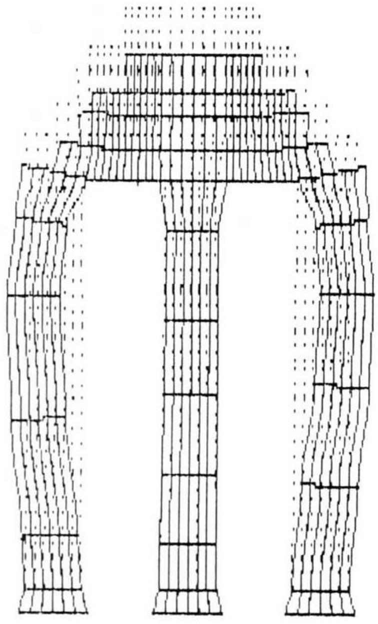


fig. 5 Tempio di Castore e Polluce: photo of the monument and of the Finite Elements model.

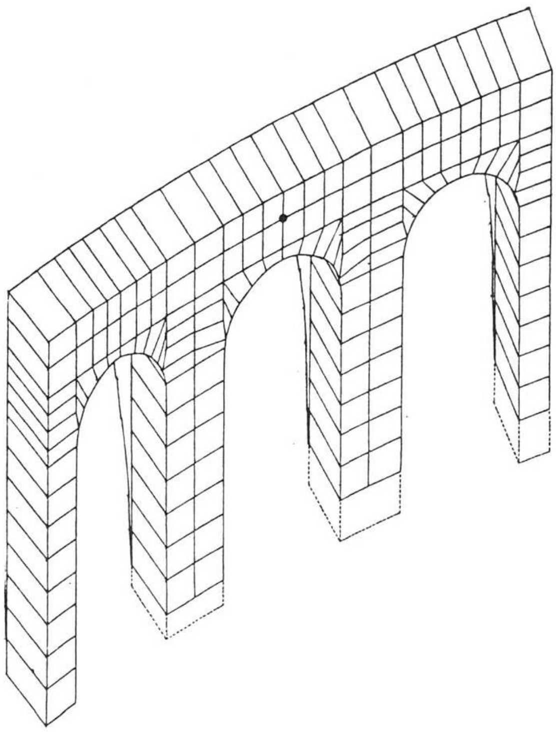


fig. 6 Colosseo: Finite Elements model of the research area.

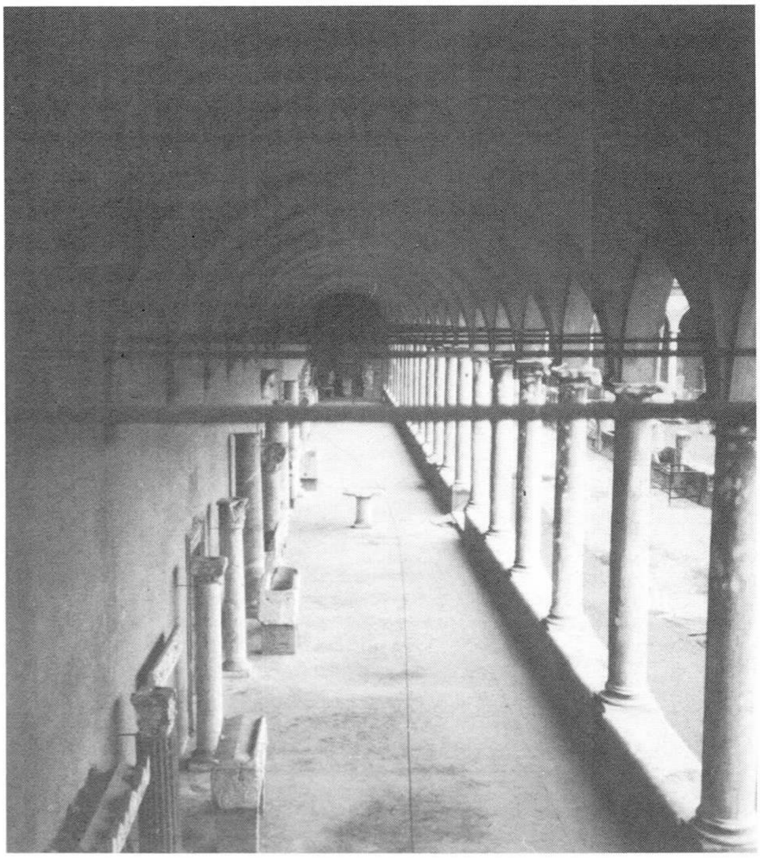


fig. 7 Photo of catenary system in Michelangelo cloister in the church of S. Maria degli Angeli in Rome.

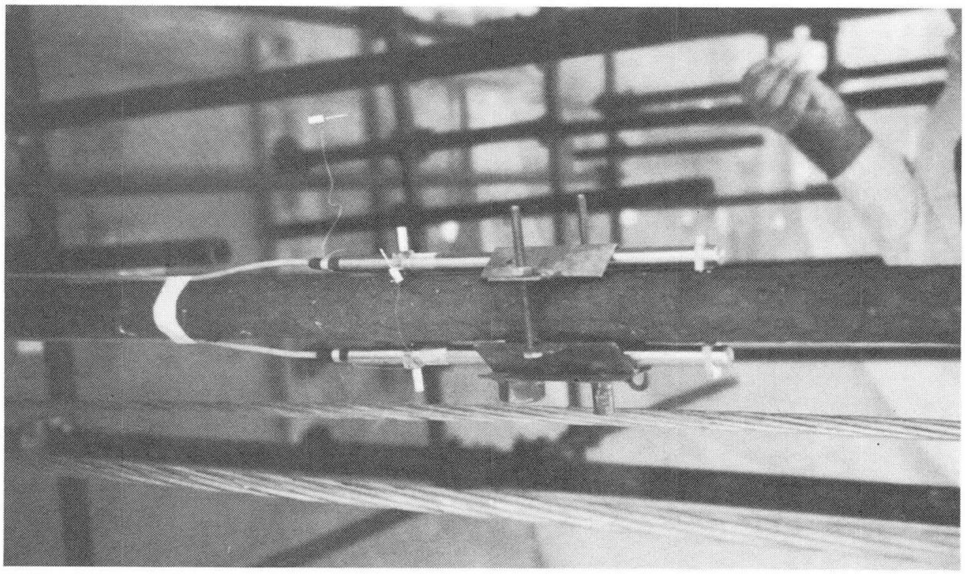


fig. 8 Photo of deformometers applied on a chain for dynamic tests.

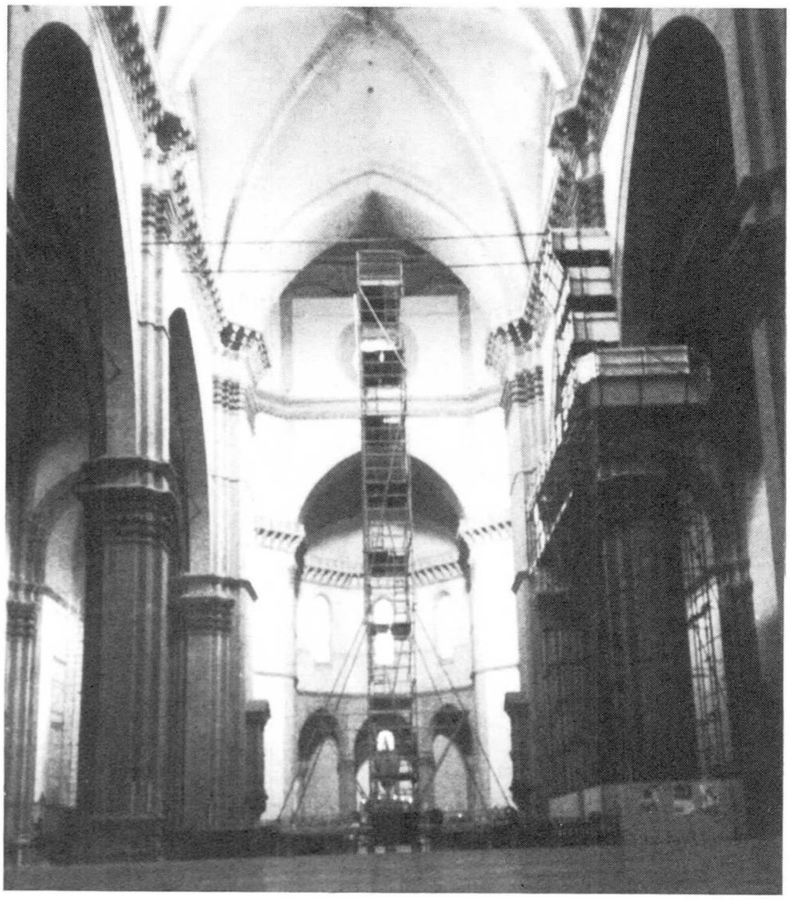
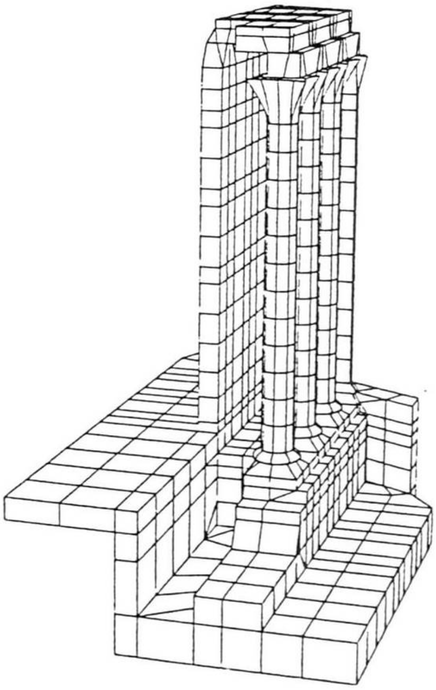


fig. 9 Photo of the church of S. Maria del Fiore in Florence during the test effected on transversal chain.

fig. 10 Claudio's aqueduct at Quadraro.



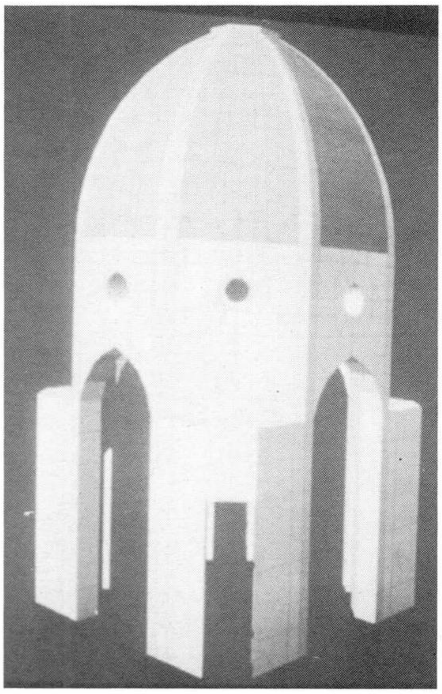


fig. 11 Tempio di Marte Ultore and Brunelleschi's Dome numerical models (CRIS-ENEL)

MONUMENT	KIND OF EXCITEMENT	SURVEY SYSTEM	SURVEY DATA	NUMERICAL MODEL
Tempio di Castore e Polluce	impulse	Acceler. Vibrom. Extensom.	eigenmodes	F.E. non linear model
Tempio di Vespasiano	impulse	Acceler. Vibrom. Extensom.	eigenmodes	non linear model
Tempio di Saturno	impulse	Acceler. Vibrom. Extensom.	eigenmodes	non linear model
Colonna di Foca	impulse vibrodina	Acceler. Vibrom. Extensom.	eigenmodes trasf.func.	non linear model
Colosseo	impulse	Acceler. Vibrom. Extensom.	eigenmodes	F. E.
Chiostro di S.M. degli Angeli	impulse	Acceler. Vibrom. Extensom.	eigenmodes	cont. model
Chiesa di S.M. del Fiore	impulse	Vibrom. Extensom.	eigenmodes	cont. model
Cupola del Brunelleschi	wind	Acceler. Vibrom.	eigenmodes trasf.func.	E.F.
Torre di Arnolfo	vibrodina	Vibrom. Laser	eigenmodes trasf.func.	-
Claudio's Aqueduct	train	Acceler. Vibrom.	frequency	-

Tab. 1 Dynamic experimental tests effected with the cooperation of the technicals of the Dipartment of Civil Engineering of Florence

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Structural Integrity of Bridge Ulenbergstrasse in Düsseldorf

Aptitude au service du pont Ulenbergstrasse à Düsseldorf

Gebrauchsfähigkeit der Brücke Ulenbergstrasse in Düsseldorf

Gert KÖNIG
Professor Dr.-Ing.
Institut für Massivbau
Technical Unversity Darmstadt
FRG

Atilla ÖTES
Dr.-Ing.
König und Heunisch
Consulting Engineers
Frankfurt, FRG

Gerald GIEGERICH
Dipl.-Ing.
Institut für Massivbau
Technical University Darmstadt
FRG

Hans J. MIESSELER
Dipl.-Ing.
Strabag Bau AG
Köln, FRG

SUMMARY
The Bridge Ulenbergstraße in Düsseldorf is the first road bridge construction where prestressing is entirely provided by resin bonded glass fibre (Polystal) tendons. In order to verify the structural integrity under service conditions a long term monitoring program has been established. In the paper, integrity monitoring techniques applicable to concrete bridges are briefly reviewed and the special monitoring program for Ulenbergstraße is presented. Finally, the results obtained until now are discussed.

RESUME
Le pont Ulenbergstraße à Düsseldorf est la première construction de pont ou la précontrainte est fournie par des éléments de précontrainte en fibre de verre collés à résine (Polystal) entièrement. Afin de vérifier l'aptitude au service sous contrainte de service, un programme de contrôle sur plusieurs années a été établi. Les techniques applicables aux ponts en béton sont révisées et le programme de surveillance de l'aptitude au service du pont Ulenbergstraße est présenté. Les résultats obtenus jusqu'ici sont discutés.

ZUSAMMENFASSUNG
Die Brücke an der Ulenbergstraße in Düsseldorf ist die erste Straßenbrücke, bei der die Vorspannung gänzlich durch die Spannglieder aus dem Faserverbundwerkstoff Polystal aufgebracht ist. Um die Gebrauchsfähigkeit der Brücke über längere Zeit beobachten zu können, wurde ein entsprechendes Inspektionsprogramm aufgestellt. In dem Beitrag werden die auf Betonbrücken anwendbaren Überwachungsmethoden kurz besprochen und das spezielle Inspektionsprogramm für die Brücke Ulenbergstraße vorgestellt. Abschließend werden die bisher gewonnenen Ergebnisse diskutiert.

1. INTRODUCTORY REMARKS

The roadbridge Ulenbergstraße in Düsseldorf has been the first major bridge project of the joint venture Strabag Bau-AG and Bayer AG, where prestressing is entirely provided by resinbonded glassfibre tendons /1/.

Prior to the execution of the project in summer 1986 comprehensive investigations have been carried out on both the glassfibre material and the special questions regarding its application to concrete bridges. The material tests did not only deal with strength and stiffness properties but also focused on aspects like resistance to chemical attack and to physical damage. Regarding the application of the glassfibre material to prestressing tendons several questions pertaining to bond properties, resistance to the perpendicular pressure in anchorages, etc. had to be solved. Finally, an appropriate design concept has been developed to cope with the special features of the new material such as the relatively low modulus of elasticity ($E = 51000\ N/mm^2$) as well as the brittle type failure and the lower long-term strength.

The tests, in principle, proved the developed material to be adequate for prestressing of concrete structures. Nevertheless, for its first major application to a roadbridge with heavy traffic loads a systematic multiannual inspection program has been deemed to be indispensible.

In principle, the inspection aims at the identification of any change of the structural integrity in an early stage. Particular emphasis is laid on the inspection of the behaviour of the glassfibre tendons in order to evaluate their fitness for the purpose. In order to meet these objectives several monitoring and identification techniques have been incorporated. However, it should be remarked, that some of these methods currently is in research stage and necessitate detailed studies to be further developed.

2. INTEGRITY MONITORING OF CONCRETE BRIDGES

The ultimate objective of the regular integrity monitoring of concrete bridges is to identify and to quantify changes in the load carrying capability. Since cracks may significantly impact the stiffness as well as the durability of a bridge the attention is primarily directed towards monitoring of cracks and detection of their sources.

Regular visual inspection may help to identify cracks if they are visible. The human eye, however, does not have access to each part of the bridge. Consequently some cracks, e.g. in top of the sections on supports in multi-span bridge systems, may remain undetected under the paving. Not only the identification of unvisible cracks but also questions like

- Which are the sources of cracking?
- Which are the residual values for stiffness and loading carrying capacity of the structure?

call for additional monitoring tools. The answers to the above questions may urgently be needed to evaluate the structural integrity and in case to initiate measures for strengthening and repair.

A monitoring technique which is nondestructive and capable to recognize both the damage as well as its source in an early stage and to quantify it in a reliable way would constitute a perfect inspection tool. However, considering the appreciable size of a bridge structure, the diversity and inhomogenity of the materials and the various environmental effects such as traffic flow, temperature, humidity, etc., it can easily be recognized that monitoring of a concrete bridge is not only associated with expenses but also with serious limitations and uncertainties.

A further basic problem in damage source detection stems from the fact that the measured quantities, in general, pertain to physical effects which result from the damage source but not necessarily reveal it directly. For example, the rupture of a prestressing tendon may lead to such a local increase of tensile stresses that concrete cracks but the rupture itself may still remain unrecognized.

In addition to these general problems also some special phenomena like the interaction of the superstructure with nonstructural and substructural elements may pose further serious questions regarding the interpretation of the measured data.

Despite all these difficulties techniques developed for integrity monitoring of concrete bridges should satisfy the following requirements:

- predicative
- objective and reproducible
- reliable
- nondestructive

and

- economical.

In the following, the potential monitoring techniques applicable to integrity monitoring of concrete bridges are discussed in the light of the above listed criteria:

- Visual inspection:

  Performed by well-trained human eye it is a highly reliable inspection technique and is inexpensive. If carried out at regular intervals the crack propagation can be recognized. However, the method is neither capable to give clear indications on the source mechanisms nor does it provide any direct means to evaluate the residual integrity quantitatively. Furthermore, it can be viewed to be useless for cases with brittle-type failure.

- Deformation and strain measurements:

  Sensitive measurements of the deformations under dead loads along both the bridge spans as well as the bridge sections carried out at regular intervals may indicate changes of the overall stiffness properties.
  To eliminate the influence of the environmental effects, special measurements under well-defined static loads can be carried out. Comparing the measured data with analytical results, the overall stiffness properties can be quantified.
  Since the deformation increase reflects an integral value of the contributions of all bridge sections, the method doesn't offer a direct means to localize the cracks effectively. It may well give indications on potential crack zones but does not allow to quantify the cracks individually. Hence, it is not suitable for identification of crack source mechanisms.
  The capability of deformation monitoring can considerably be improved by strain monitoring. Concrete surfaces in critical zones, prestressing tendons

or reinforcing bars can be effectively monitored using appropriate strain gauges. In connection herewith, also the use of load cells should be mentioned. For example, monitoring the prestressing force in a tendon, which is not bonded with concrete, by using a load cell can be helpful to recognize changes in the overall behaviour.

- Vibration monitoring (dynamic system identification):

Systematic vibration monitoring aiming at the identification of dynamic parameters such as natural frequency, damping and modal shapes may offer substantial means to quantify the stiffness changes and to localize their sources.
If events like overloading, restraint or loss of prestressing result in cracks or high stress concentrations, stiffness and damping properties of the vibrating system change. With increasing damage the natural frequencies decrease wheras the damping values increase. Furthermore, modes which have large relative rotations in damaged zones exhibit a higher relative change in their eigenfrequencies and thus give indications on potential damage zones. Since many mode shapes can be excited and checked with regard to their properties, for the detection of damages dynamic monitoring provides a much larger data basis than the static displacement measurements.

Vibrations can be induced either by artificial excitation or by ambient vibrations. In any case, system identification tests are conducted at very low-level excitations since testing may not cause any damage to the structure. Hence, sensitive transducers are needed to capture the dynamic response of the structure.
Having identified the dynamic properties of a bridge shortly after construction, a representative mathematical model can be developed by fitting its parameters to those gained from the experimental data. Evaluating the data from periodically performed measurements the eigenfrequencies, damping values and mode shapes can be determined and compared with the reference values of the first measurement (nonparametric identification). If changes are identified, adequate methods must be implemented to fit the mathematical model to the new data (parametric identification) /2/. This step is necessary to determine the change of structural properties such as stiffness, mass and stresses which are essential to draw conclusions concerning the state of the structure.

For an effective use of this monitoring technique following prerequisites must be considered:

- Adequate equipment: wide-band exciter, sensitive accelerometers, highly capable data acquisition system
- For interpretation and pattern recognition: profound knowledge on nonlinear force-deflection characteristics of R/C bridges with emphasis on cracking and its effects on stiffness and damping properties.
- Additional measurements and experience in pattern recognition for environmental, nonstructural and substructural interactions.

The current state of practice is such that considerable additional work is necessary for both to further develop the identification technique as well as to determine the limits of application to bridges quantitatively.

- Monitoring using optical sensors:

Inaccessible parts of the bridges, e.g. the prestressing tendons can be effectively monitored by means of optical fibres. If appropriately attached to the tendons or even integrated in the tendons as in the case of glass-fibrons tendons, the optical fibres may offer the possibility to observe the tendon deformations remotely. If appropriately designed and reliably measured, the fibres can serve a dual purpose: First, they indicate the elongation of the prestressing tendon by the decrease of the transmissibility of the light, which is due to the fact that the transverse strains reduce the fibre section. Secondly, fracturing of the fibre can either indicate a given stress level or the fracture of the tendon, both causing a loss of the light at the detector. Measuring the reflection, the location of the fracture can be determined.
Problems may arise in connection with transverse stresses in the anchorage zones as well as at the bends of the tendons. Current studies are quite promising /3/.

- Crack monitoring by acoustic emission analysis:

Microcracking, local deformation, friction and plastification in inaccessible parts of a concrete structure, in principle, can be captured detecting the stress waves released from such sources by piezoelectric sensors /4/. Parameters such as energy, wave amplitude, duration and frequency content may comprise useful information about the source mechanisms. Crack initiation and growth can only be captured through permanent monitoring whereas the energy release due to friction effects may also be detected by inspections at regular intervals provided that a dynamic situation exists. Questions pertaining to problems such as the environmental noise, the attenuation of the waves and the small amplitude of the stress changes are currently dealt with in systematic investigations /3/.
In any case, for a rational monitoring of a bridge the method should be considered in a later step, namely if the techniques presented before indicate that a deterioration has occured. Having a first estimate about the location of the defect the acoustic emission analysis can be implemented to detect the damage and, if possible, to measure its size.

- Ultrasonic testing

Ultrasonic testing techniques are widely used to detect cracks and material defects /5/. In concrete, however, the reduction of the transmitted ultrasound and reflected echoes is much more emphasized than in metallic materials. Because of the porosity of the material the absorption and particularly scattering may become so high that the signals for defects cannot be recognized. In order to reduce scattering wave lengths must be increased. This measure, however, results in an increase of divergence of the sound and complicates the identification of the defect size.
The frequency range for application to concrete is 50 - 150 kHz. In this range the transmission technique can be used to detect cracks and flaws by changes of the transmission velocity.

3. INTEGRITY MONITORING OF BRIDGE ULENBERGSTRASSE

Bridge Ulenbergstraße is a two-span roadbridge designed for the load class 60/30 (tons) according to DIN 1072 (Fig. 1). In the longitudinal direction the bridge is prestressed with 59 glassfibre tendons, each composed of 19 dia 7,5 mm tendon rods and providing a tensile working force capacity of 600 kN per unit.

The serviceability requirements for partial prestressing according to DIN 4227, part 1 governed the determination of the number of tendons. The amount of the ordinary reinforcement is sufficient to cope with an increase of stresses due to cracking of concrete without yielding. The ultimate load carrying capacity is 30 % higher than required by the design provisions. Additional safety is provided by nonprestressed steel tendons which are conceived as "emergency belts". In case of large deformations of the structure they would automatically be activated and would contribute to the load carrying capacity. Further, space is preserved for additional steel tendons to be installed at an advanced level of damage in order to suppress the cracks and thus to rehabilitate the bridge /5/.

Under service conditions no cracking is supposed to occur. The analysis shows that quite a number of prestressing tendons may fail without causing a visible crack in concrete. This specific aspect clearly states the need for a monitoring tool for early warning.

Since the modulus of elasticity of glassfibre is much lower than of steel, cracking is assumed to result in larger deformations exhibiting wider crack openings. Test on beam specimens have clearly confirmed this phenomenon. For the inspection task this fact certainly means a simplification but considering the consequences of large cracks one can easily realize that a quick and effective source detection is of paramount importance. Hence, adequate monitoring techniques are needed which are capable to evaluate both the magnitude and the consequences of the damage. In connection herewith, the review of design assumptions and further analytical investigations, within certain limits, may help verify the quantitative findings by the measurements.

Finally, for the advanced level of a damage where repair or strengthening measures must be taken, effective methods are needed to answer questions like whether the damage in the tendons is a local phenomenon or whether it is propagating or not.

Considering the above design aspects as well as the conceivable damage patterns and levels a multiannual monitoring program has been set up. The monitoring techniques chosen are viewed to support each other (Table 1). In the following, for each of the monitoring techniques the special aspects regarding the implementation and the results obtained from the measurements until now are presented:

- Visual inspection: No cracking could be observed during the visual examinations carried out hitherto.

- Deformation measurements: The vertical displacements under dead loads and well-defined truck loads (2 x 22 = 44 tons) have been measured in 24 points using a level. 2 x 9 points are located on the superstructure reflecting the bending modes in longitudinal as well as perpendicular directions. On each of the supports two points have been monitored in order to separate the relative deformations from the overall displacements. The accuracy of levelling performed until now is approximately 1 mm which is of the same order as the

maximum displacements under truck loads. Since the geometry of the prestressing tendons is such that no compressive forces act at the level of the tendon, creep effects on displacement can be neglected. In order that the temperature effects remain negligable, the measurements have been carried out at about 5 a.m.
Until now the measurements did not indicate any change of the deformation characteristics.

- Monitoring of unbonded tendons: Permanent measuring of the prestressing force in three unbonded glassfibre tendons by load cells provides a further tool to recognize major changes in the structural behaviour. Since the force change is controlled by the global deformations of the bridge, local changes cannot be captured sensitively. However, any integrity change of the prestressing tendon itself can easily be captured.
  Until now no indication on integrity loss could be observed.

- Dynamic system identification: Prior to the application of this method various analytical models for the bridge superstructure have been studied with regard to modal parameters such as natural frequencies and mode shapes. A simple beam model has been quite sufficient to reflect the bending modes but because of the high width/span ratio of the superstructure it has beennecessary to develop a 3D-model which captures the geometry entirely.
  Based on these analytical estimates the optimal locations for artificial excitation as well as measurement points have been established. For the excitation such positions have been selected which are far enough from the contraflexural sections of all the modes of interest. For the measurement points an appropriate grid size has been chosen which offers the possibility of capturing of all the modes effectively (Fig. 2). In meeting the decision about the number of the measurement points also economical aspects have been considered, since only a limited number of transducers are available.
  The bridge has been excited dynamically using a hydraulic actuator mounted on a concrete block and placed on the superstructure (Fig. 3). Accelerating the mass (460 kg) on the jack with a random noise in the range of 0 - 64 Hz, vertical forces have been applied to the structure. In order to avoid any damage to the structure the force level has been limited to 15 kN. Some of the basic criteria which led to this type of excitation are:

  - As the number of available transducers is significantly smaller than the number of measuring points, an excitation technique is needed which is capable of exciting a wide range of frequencies and can reliable be reproduced multiple times. The latter feature is essential to satisfy the requirement for constant excitation during the stepwise identification of the entire measuring grid.
  - The applied force can be kept almost constant over the entire frequency range and be controlled easily. In case, the level of the forces as well as the range of frequency can be adjusted to the specific boundaries to achieve a better identification.

  The dynamic response of the bridge has been recorded using sensitive seismometers wherby in each configuration three measuring points and one reference point (top of exciting mass) have been recorded. The signals have been amplified and transferred to a 4-channel FFT analysator. Autospectra and transfer functions have been calculated and stored. Using the software package MODAL PLUS the modal values of the structure have been obtained (Table 2). In Figure 4 the corresponding mode shapes are presented.

Referring to the results summarized in Table 2 following statements can be made:

- The comparison between the first and second measurements clearly shows an increase of the natural frequencies. The increase in modes corresponding with bending are smaller than in modes associated with torsional modes. The increase in general can be explained by the ongoing increase of the Young's modulus of concrete after construction as well as by the reduction of mass due to loss of water in concrete. Furthermore, since the loss of water in the cantilever parts may be more pronounced, the torsional modes reflect a higher change of eigenfrequencies.

- The discrepancy between the results derived from the measurements and those obtained by analytical models is partly due to the fact, that the mathematical model is not an ultimately refined model. Nevertheless, uncertainties associated with the simulation of nonstructural elements, bearings and interactions with the substructure are viewed to pose serious questions with regard to analytical prediction as well as with regard to interpretation of any measured change in the structure.

It can be concluded that both procedures, namely the evaluation of the measured data as well as the mathematical modelling including the parametric identification necessitate further thorough work. Studies on damage assessment modelling are planned to investigate the relationship between the damage parameters and the modal properties. The outcome will help to quantify the sensitiveness of this monitoring technique.

- Integrity monitoring using sensors:

  For the purpose of monitoring the glassfibre tendons in a direct manner, optical sensors have been used. Three arbitrarily chosen tendons have been equipped with special lightwave-conductors developed by Felten & Guilleaume, Köln (Fig. 5). As briefly described in the preceeding chapter, any change of the axial strain in the conductor also results in a change of the fibre section. The transverse compression is amplified by using a spiral wire wrapped around the conductor. If the glassfibre tendon is subject to elongation, the section of the optical fibre is reduced under the pressure of the wire and as a consequence of this the transmissibility of light decreases (damping effect).
  Tests clearly proved that the relationship between elongation and damping is characterized by a constant value independent upon the number of test cycles (Fig. 6). Furthermore, through implementation of the impulse reflection technique the location of any rupture in the fibre can be determined with an accuracy of $\pm$ 10 cm.
  One further tendon has been equipped with an alternative sensor type, a copper wire, the capacitance of which can be measured and used for the identification and location of a possible rupture.
  Investigations on both sensor techniques are underway within the framework of an integrated research effort (BRITE) aiming at development of nondestructive techniques for concrete structures /3/.
  Until now none of the sensors gave any indication on a significant change of the force and geometry of the tendons they are embedded in (Fig. 7).

- Acoustic emission analysis

This method is also dealt with as a subproject within the BRITE research program. Systematic investigations focussing on parameters for damage detection are currently carried out by "Fraunhofer Institut für zerstörungsfreie Prüfverfahren" in Saarbrücken.
Since this monitoring technique, in principle, is conceived for implementation in an advanced level of damage, so far no need has been recognized for application to the Bridge Ulenbergstraße.

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TIME (quarter year)	Completion of construction	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Visual inspection	Initial measurement	x	x	x	x	x	x	x	x		x		x		x		x				x				x
Deformation measurements	"	x	x		x				x				x												x
Dynamic monitoring	"	x	x		x				x				x												x
Monitoring of unbonded tendons	"	x	x	x	x	x	x	x	x		x		x		x		x				x				x
Monitoring using sensors	"	x	x		x				x				x												x

Table 1 Monitoring program for Bridge Ulenbergstraße

Mode No.	2D-Model (Beam elements) for the bare	3D-Model (Shell elements) superstructure	1st Measurement	2nd Measurement	Mode type
1	4.45	4.412	4.95	5.11	bending
2	7.71	7.661	7.07	7.42	bending
3	--	13.238	9.92	10.16	torsion
4	--	16.257	11.53	12.46	bending in transversal direction
5	17.08	16.836	18.94	20.79	- " -
6	25.27	24.153	21.62	22.55	bending + torsion

Table 2 Eigenfrequencies (Hz)

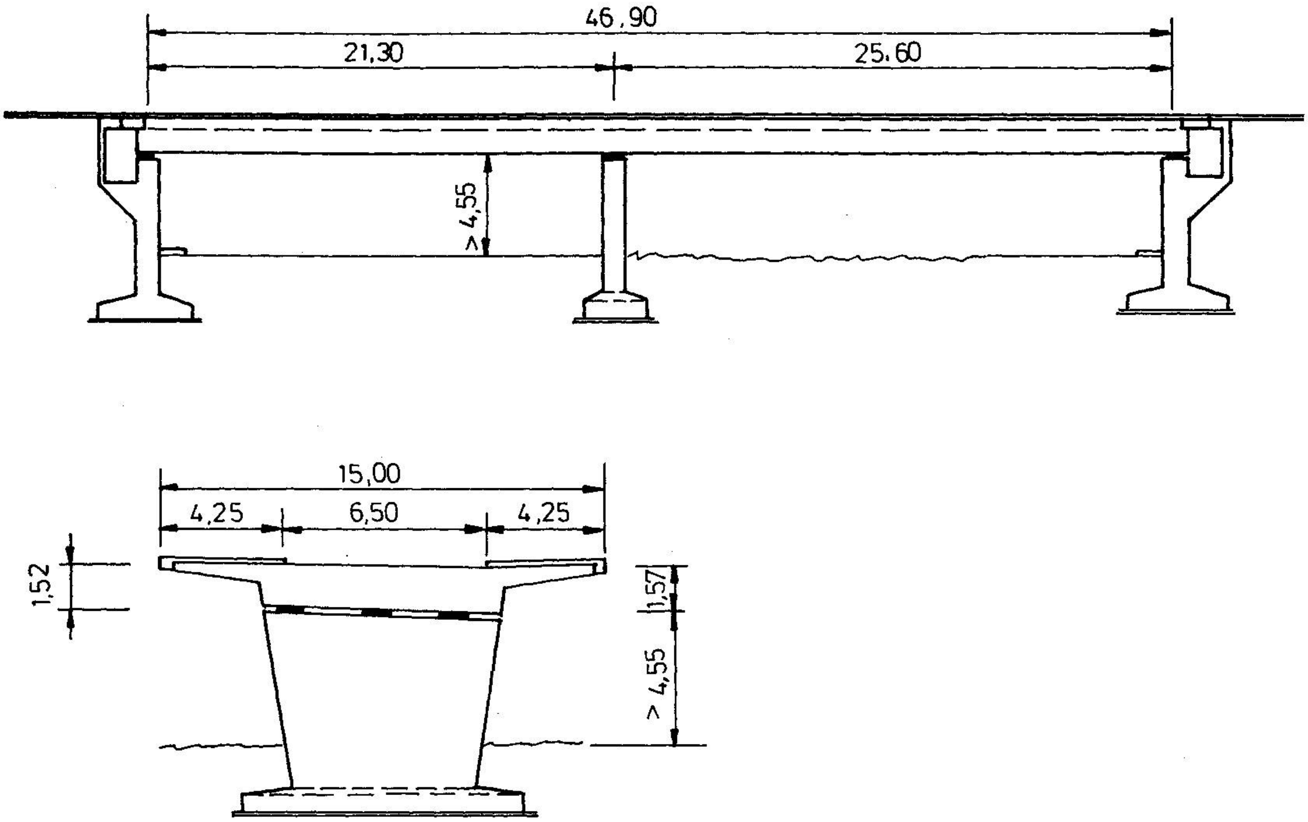

Fig. 1 Bridge Ulenbergstraße, Longitudinal and cross sections

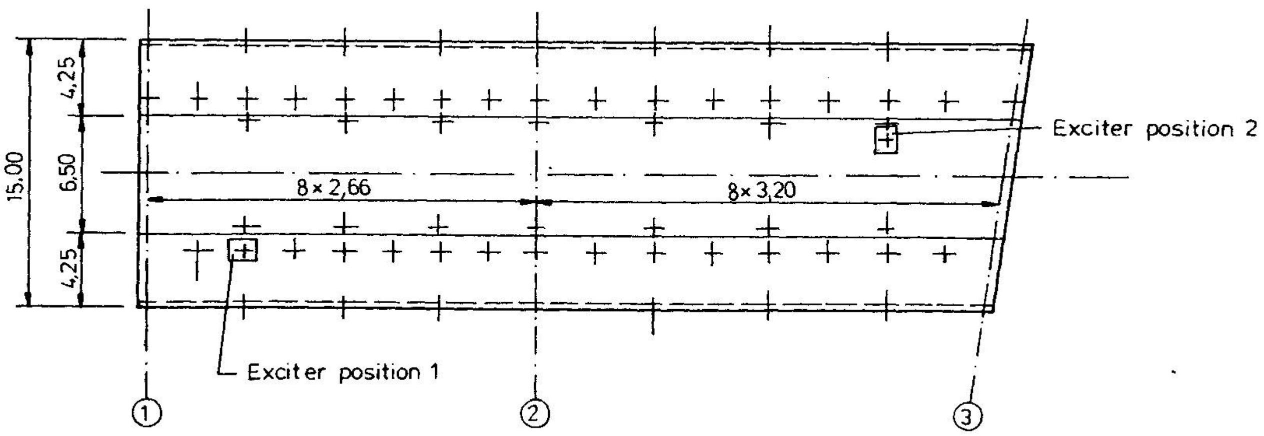

Fig. 2 Bridge Ulenbergstraße, Plan view
Exciter positions and measuring grid

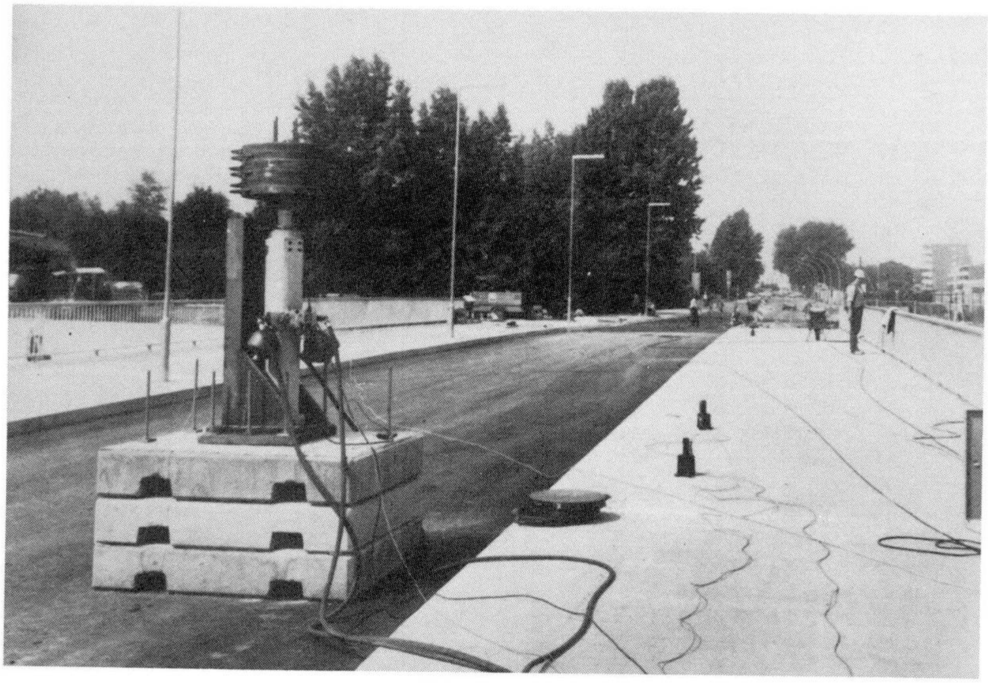

Fig. 3 Bridge Ulenbergstraße, hydraulic exciter (1st measurement)

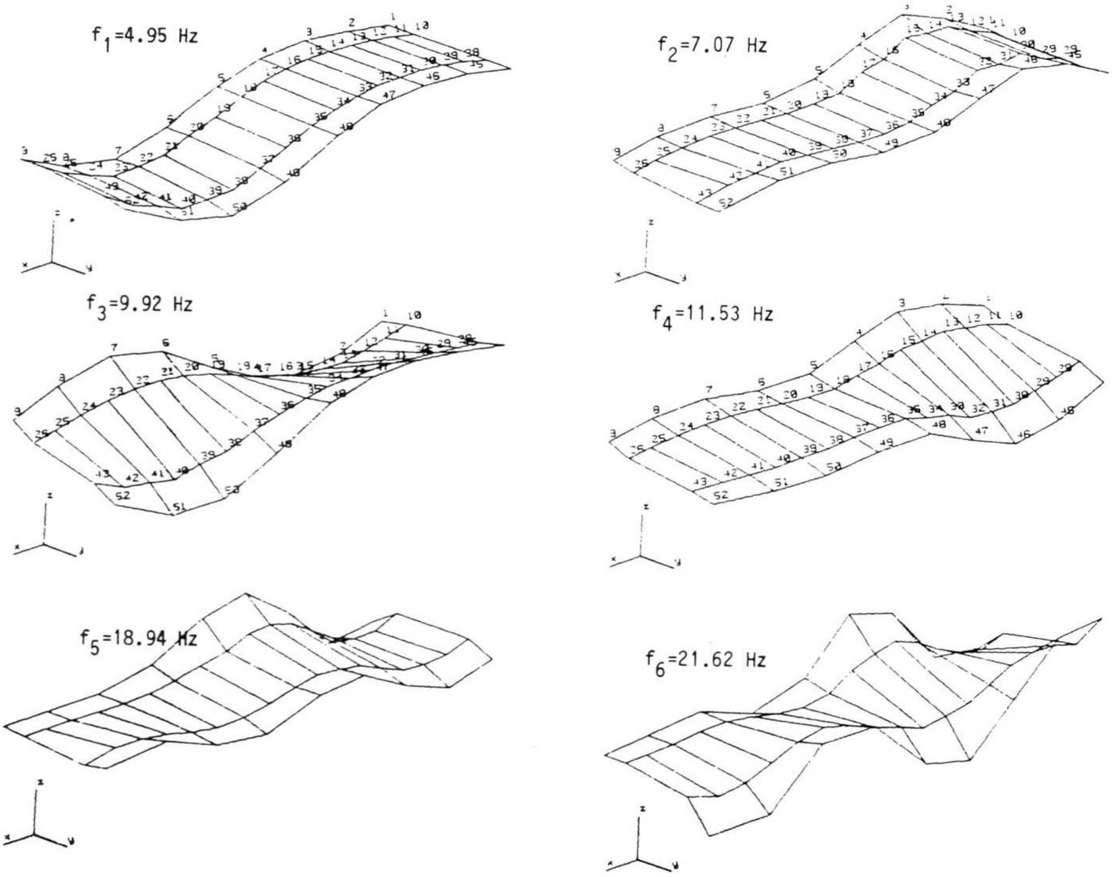

Fig. 4 Mode shapes extracted from the measured response

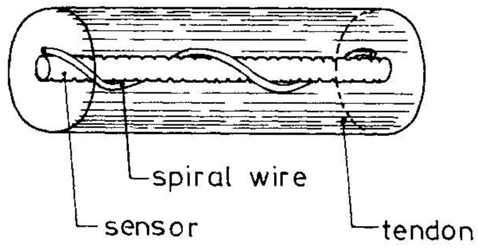

Fig. 5 Optical sensor (Felten & Guilleaume)

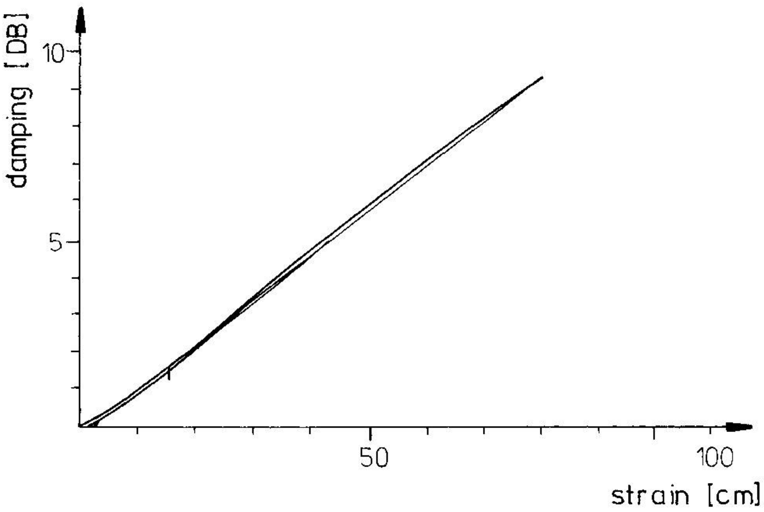

Fig. 6 Damping-strain relationship of the optical sensor

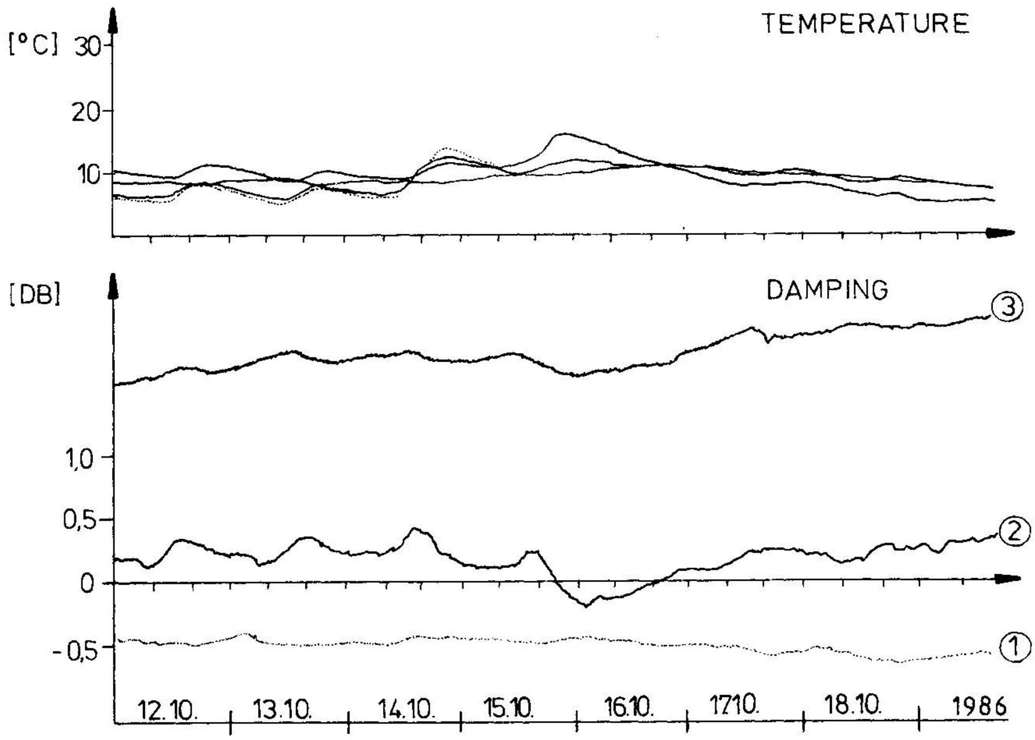

Fig. 7 Optical sensor monitoring on Bridge Ulenbergstraße

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Control and Detecting System related to the construction of the underground railway line in Rome

Système de contrôle et de détection établi pour la construction du métro de Rome

Für den Bau der Römer Untergrundbahn aufgestelltes Überwachungs- und Entdeckungssystem

Mario CANGIANO
Civil Engineer
I.M. Intermetro S.p.A.
Roma

Giorgio CROCI
Professor
University of Rome
Rome, Italy

Mario Cangiano, born in Roma in 1941, graduated at Rome University in Hydraulic Engineering in 1968. For three years he had experience in road yards. Since 1971 he has been interested in the realization of the underground railways. At the moment he is clerk of works for line «B» extension of Rome Underground for I.M. Intermetro (Main contractor).

Giorgio Croci, born in 1936, received his civil engineering degree and is now professor of Tecnica delle Costruzioni with the Faculty of Engineering at the University of Rome. He has carried out a large number of projects for strengthening ancient monuments and has organized courses and conferences on the subject.

SUMMARY
The paper presents, in connection with an application to some old masonry buildings in Rome, the criteria for the control of the behaviour of a structure affected by external works (excavations, back fill, near constructions, etc.). The control involves the use of an instrumentation and monitoring network and has a double function. It is in a sense an «insurance policy»; it can represent, in fact, a warning bell that intervenes only in case or unfavourable of dangerous events. It can provide a knowledge of the actual response of a new or old structure to imposed foundation settlements.

RESUME
Cet article présente les critères à la base du contrôle du comportement d'une structure affectée par des travaux extérieurs (excavations, remblayages, constructions voisines, etc.) ainsi que leur application à quelques anciennes constructions en maçonnerie à Rome. Le contrôle comporte l'utilisation d'un réseau d'instruments et d'une châine de surveillance ayant une double fonction: il s'agit dans un certain sens d'une «police d'assurance» et consiste en fait en un signal d'alarme intervenant seulement en cas d'évènements défavourables ou dangereux. Le systèmes en question permet de relever la réponse effective d'une nouvelle ou ancienne construction à des tassements imposés.

ZUSAMMENFASSUNG
Dieser Artikel stell die, für die Überwachung von äusseren Arbeiten (Ausgrabungen, Aufschütten, benachbarte Arbeite, u.s.w.) beeinflussten Bauwerken, gründsätzliche Kriterien vor, sowie deren Anwendung für einige ältere Mauerwerkbauten in Rom. Die Kontrolle erfasst das Verwenden von Messgerät- und Überwachungsnetze und hat eine zweifache Tätigkeit: es handelt sich in einer Sinn um einer «Versicherungsvertrag», und kann eigentlich eine Alarmhupe darstellen, welche nur im Falle von ungüstigen oder gefährlichen Ereignissen eintrifft.
Dieses System ermöglicht die wirliche Antwort von neuen oder älteren Bauwerken zu gedrängte Grundstützensekungen zu ermitteln.

1. INTRODUCTION

Disturbances connected with external works (such as excavations, filling, nearby construction etc.) can cause significant alterations to the pattern of stresses in the structural elements of buildings.

Such disturbances can be represented as deformations imposed on the foundations as a result of soil subsidence.

In the case of masonry buildings, these deformations may cause lesions or the aggravation of an existing pattern of cracks, according to the stresses induced in the bearing elements.

The present article examines some old masonry buildings in Rome in the area between Via Palestro, Via Vicenza and Via Villafranca (Figures 1-3), which have been affected by works to extend Underground Line B. Two tunnels, first one, then the other, for this extension were bored under these buildings at a depth of 20-25 metres below the road surface

The masonry structures examined (see para 2) were markedly cracked prior to the start of work on the tunnels. When the second of the two tunnels was built, it was decided, in conjunction with the Intermetro and La Girola companies, to adopt a system to check the structures concerned.

The check was based on the use of an instrument network to provide data on deformations and therefore stresses in the materials, the width of cracks, lesions, joints, temperatures, movements of foundations, and water levels. These data were acquired automatically and in real time.

In both this specific case and also generally, the control system has a dual role. On the one hand it is a sort of insurance policy, money spent to ensure that people can use the buildings without incurring risks, at the same time keeping preventive measures to the minimum, since each and every alarm signal is transmitted promptly and allows any necessary measures to be taken in time (such as evacuation, buttressing, reinforcement, etc.).

It also makes it possible to know the real response of the structure to the deformations imposed at the base of the foundations. This may be very useful, both in correctly analysing the development of the crack picture and in rational definition of the consolidation works necessary, and also as a test for use in developing methods for use in similar buildings to which the results may be extrapolated.

2. DESCRIPTION OF THE WORKS AND PRE-EXISTING CRACK PATTERN

The buildings to which the recording system described in this paper was applied comprise the end part of a block, situated between Via Palestro, Via Vicenza and Via Villafranca and arranged in the form of a "U".

The three buildings recorded are situated one against the other and separated by joints in the construction (see Figures 4-7). The characteristics of the three buildings are very similar and can be summarised as follows.

The buildings comprise a basement part-floor, a ground floor mostly used as shops, and 5 or 6 floors over. The roofs are flat, terrace-type.

The buildings were constructed in the early 1900s, and entirely of masonry. The thickness of the vertical structures is 80-90 cm at ground floor level, decreasing to 50-60 cm at the top floor.

The external width of the window openings is about 100-120 cm on all floors except the ground floor, where the windows are about 2 m wide. The height of the windows, however, varies according to the floor. The foundations are of hardcore-filled "wells" connected by masonry arches on which the wall rest. The stairs are mostly "Roman" type (resting on masonry half-arches) with steel beams and slabs introduced later in some cases. In general the horizontal structures are small arches resting on steel beams at about 85 cm spacing.

Fig. 1 Facade looking on Via Palestro

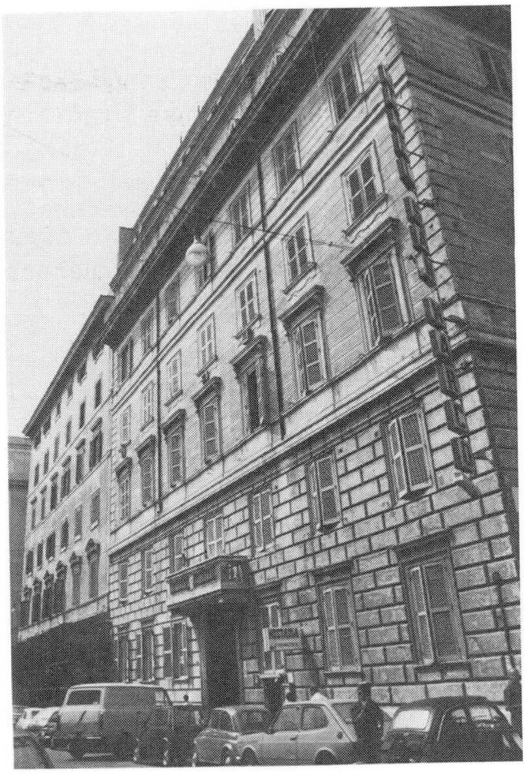


Fig. 2 Facade looking on Via Vicenza

Fig. 3 Facade looking on Via Villafranca

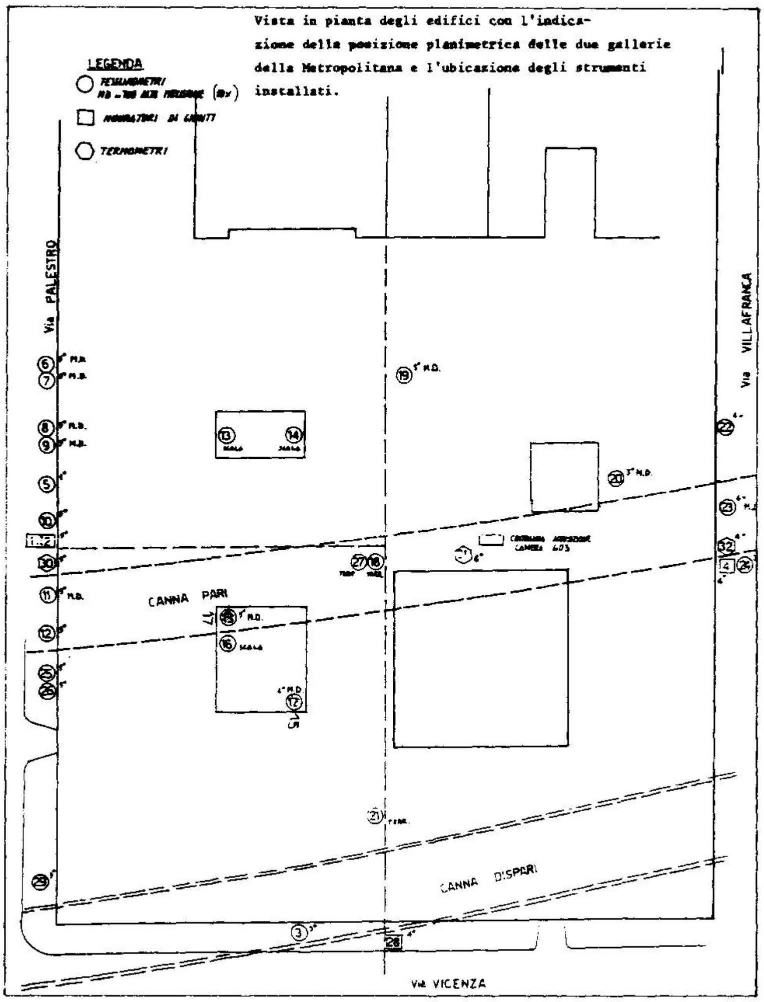


Fig. 4 Plan view of the buildings with the indication of the two underground tunnels position and of the instrument location.

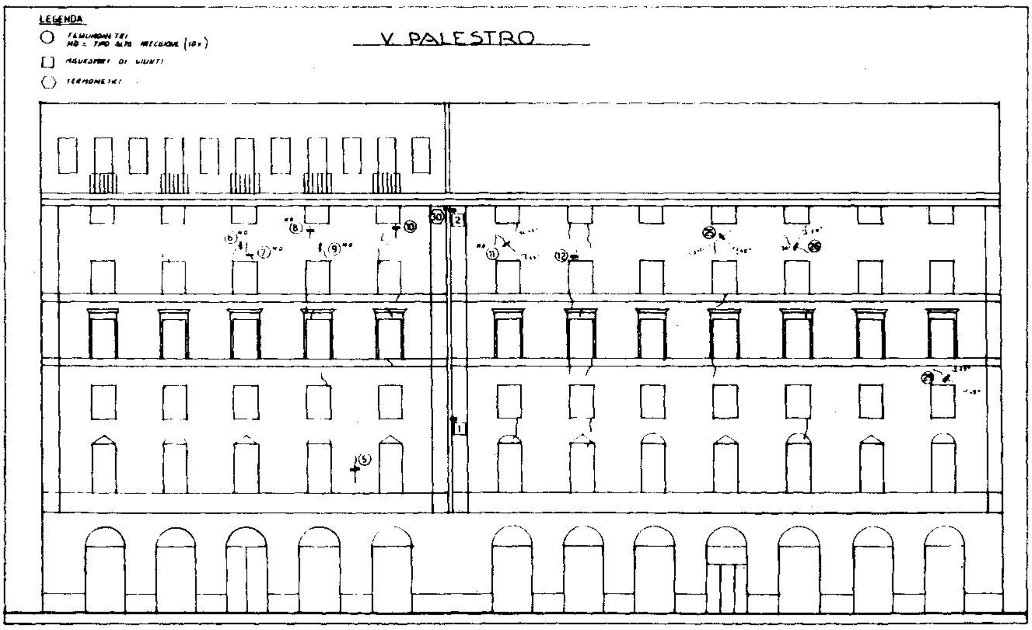


Fig. 5 Facade looking on Via Palestro
The crack pattern and the installed instruments are visible.

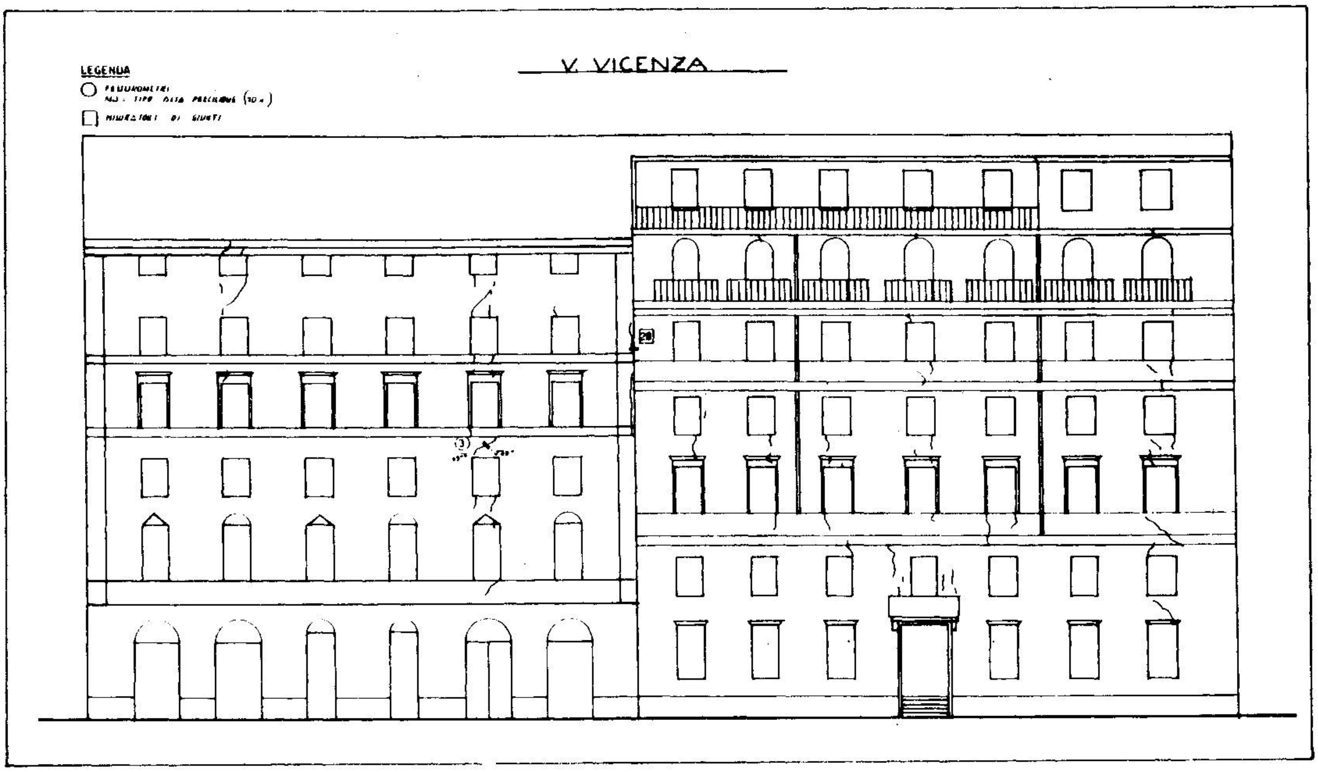


Fig. 6 Facade looking on Via Vicenza.
The crack pattern and the installed instruments are visible

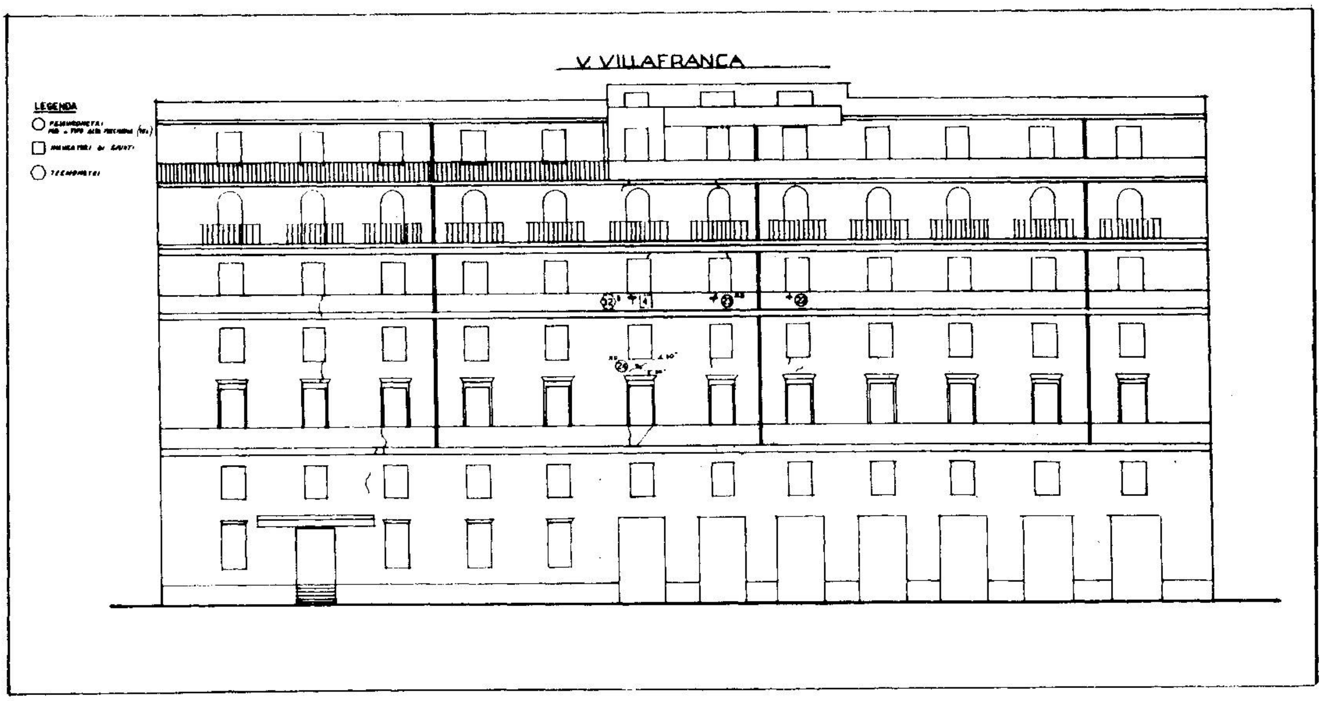


Fig. 7 Facade looking on Via Villafranca
The crack pattern and the installed instruments are visible

Some structures in the buildings in question have been modified at various times. Both the outsides and the internal structural elements show pronounced craking. The lesions present on the outsides are shown in Figures 5-7, while Figures 8-11 show the most significant lesions encountered inside the buildings.

This pattern of craking, which existed prior to construction of the second tunnel for the Underground and to the installation of the measurement system described in this article, is attributable to a series of causes and phenomena occurring during the time since the buildings were constructed. Two of these seem to be largely responsible for the present static condition of the buildings.

The first is settling of the foundations due to rises and falls in the groundwater level. This is a characteristic of the whole zone and has, over time, produced a well-defined pattern of cracks in all the buildings examined. Such effects have been aggravated by a deterioration (sometimes directly observable) in the construction materials used, not all of which were of the best quality.

The second factor is the series of minor earthquakes occurring during the century, which have contributed to and accentuated the process of deterioration.

3. DESIGN CRITERIA FOR THE INSTRUMENTATION NETWORK

The instrumentation network was designed to check the most significant lesions on both the insides and outsides of the buildings studied: on the faces looking into Via Palestro, Via Vicenza and Via Villafranca, on the terrace roofs, and on the walls facing the courtyards.

Knowledge of the change in the width of these lesions over time as excavation of the second tunnel proceeded was considered necessary for two reasons. First, to provide a continually updated picture of the static conditions of the buldings, thus providing a complex of interrelated pre-alarm signals whenever the buildings might behave in any way abnormally. Second, interpretation of the signals received from the measuring instruments, together with other data, would make possible a theoretical analysis of the behaviour of the structures studied and correlation between the results of these analyses and the experimental measurements (see para 5). The author was also given access to the data recorded by a ground engineering instrument network (set up by others) covering the whole zone and composed of subsidence meters, inclinometers and piezometers.

Figures 4-7 show the positioning of the instruments on the structures and some details are visible in Figures 12-15.

The instrumentation network installed included two thermometers positioned on the outside walls, facing Via Palestro and Via Villafranca respectively and a third located within the operations room. These made it possible to correct the other measurements for the effect of temperature variations.

4. INSTRUMENTS AND DATA ACQUISITION SYSTEM

The instruments used were as follows.

14 Crack width measuring devices (fissurometers) made by the SIS (Società Italiana per la Strumentazione Geotecnica) company and consisting of a potentiometric transducer. The field of measurement is ± 10 mm and the resolution 0.1 mm.

11 Deformation measurement deivces, SIS Mod. D312, consisting of a potentiometric sensor. The field of measurement is 0-25 mm and the resolution is 0.01 mm.

4 Monodirectional joint measurers, SIS Mod. D311, consisting of a potentiometric transducer with a plastic conductor. The field of measurement is

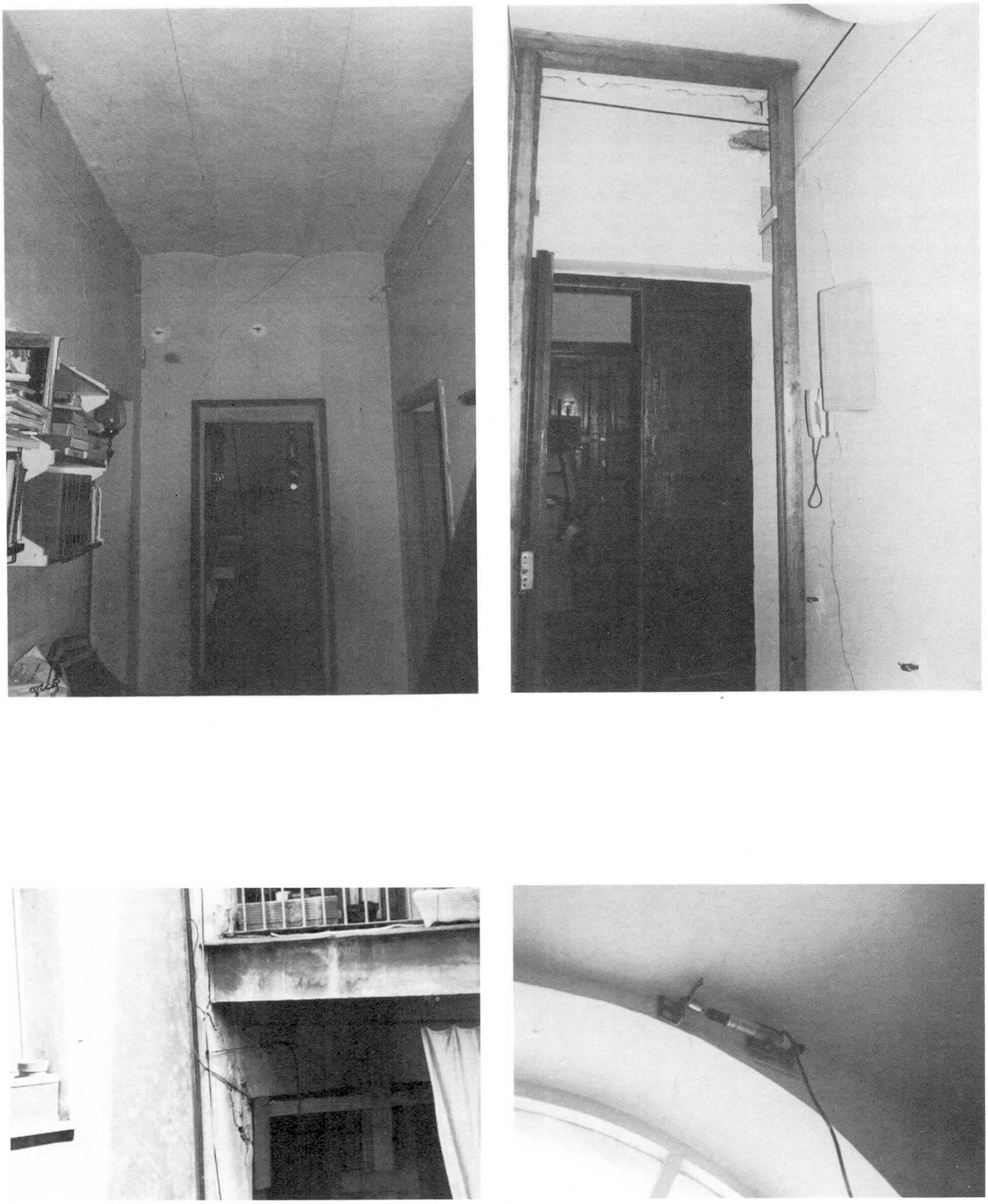


Fig. 8-11 Some of the most significative cracks on the masonry structures

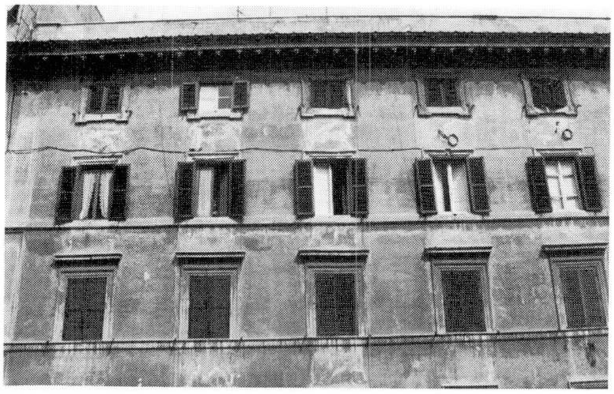


Fig. 12 Facade on Via Palestro with some of the installed instruments

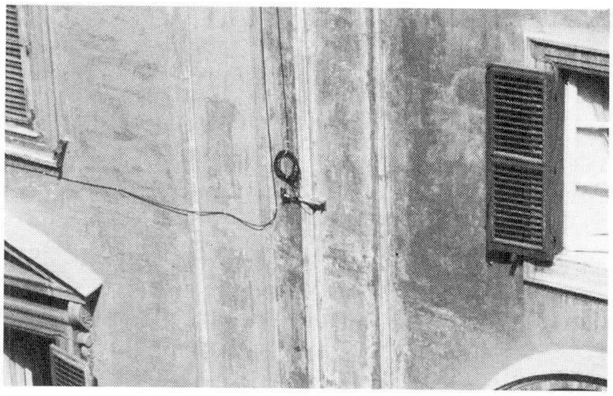


Fig. 13 The instrument n° 1

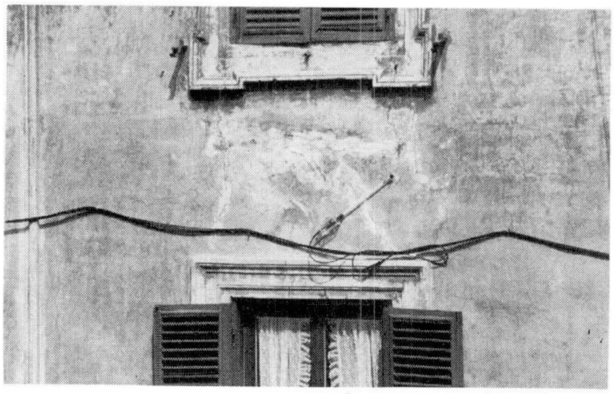


Fig. 14 The instrument n° 11

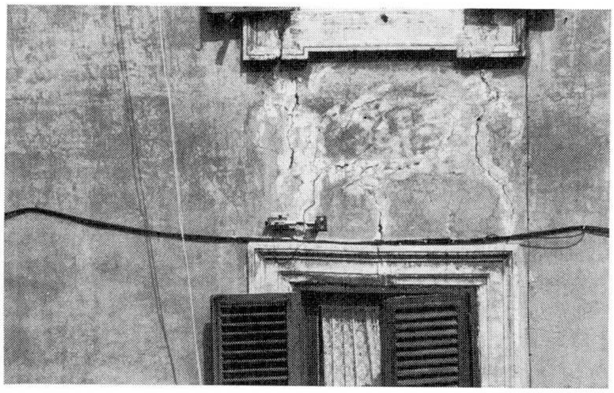


Fig. 15 The instrument n° 12

Table 1

graphics	channels	cables	instruments
1	18 19	12 25	FIUSSUROMETER
2	22 23	13 14	FIUSSUROMETER
3	1 5	16 21	FIUSSUROMETER
4	2 3	17 19	DEF. MEASUR
5	4 17	20 11	DEF. MEASUR
6	20 21	26 29	FIUSSUROMETER
7	6 7	30 31	TEMP. SENSOR
8	24	15	DEF. MEASUR
9	8	32	TEMP. SENSOR

Table 2

graphics	channels	cables	instruments
1	22 23	23 24	DEF. MEASUR.
2	17 24	27 4	FIUSSUROMETER
3	18 19	18 28	JOINT MEASUR.
4	20 21	3 22	FIUSSUROMETER
5	7 8	1 2	JOINT MEASUR
6	1 2	6 7	DEF. MEASUR
7	3 4	8 9	DEF. MEASUR
8	5 6	5 10	FIUSSUROMETER

± 25 mm and the resolution 0.1 mm.
3 Temperature sensors, SIS Mod. T111, semiconductor type, field of measurement - 10°C to + 60°C, accuracy better than 1°C.

These 32 instruments were connected to two electronic control units with 16 channels each for automatic recording of the data measured. The acquisition interval for these two units can be varied and in the period in which the checks described were carried out was varied between 4 minutes and 1 hour.

Tables 1 and 2 show the control units and channels to which the sensors were connected. The same tables show the coupling of the instruments in the graphs discussed in para 5.1.

The control units, wiring panels, computer and printer were sited in a room in the Hotel Villafranca, in one of the buildings checked.

5. INTERPRETATION AND FORECASTING OF STRUCTURAL BEHAVIOUR

5.1 Processing of data supplied by the instruments

The supplied data by the instruments were acquired and memorized by the (remote) control units. The polling interval for these units can be programmed at the start of each recording period and can also be varied at any time. The intervals used in this case varied between 4 minutes (which is the minimum and can be considered as effectively real time) and 1 hour.

The data were processed by converting the measurement units to technical units and plotting these against time. The resulting graphs could be displayed immediately on the screen of a personal computer and subsequently printed on a portable printer connected to the same computer. Figures 16 and 17 are examples of the graphs produced. The processed data were recorded on diskettes.

Later development of the data acquisition system makes it possible to present the data recorded by the instruments in graphs produced in real time, using a subsequent generation of remote control unit, special software and a personal computer permanently connected to the acquisition equipment, which is given the name of "scanner".

5.2 Analysis of structure behaviour using theoretical models

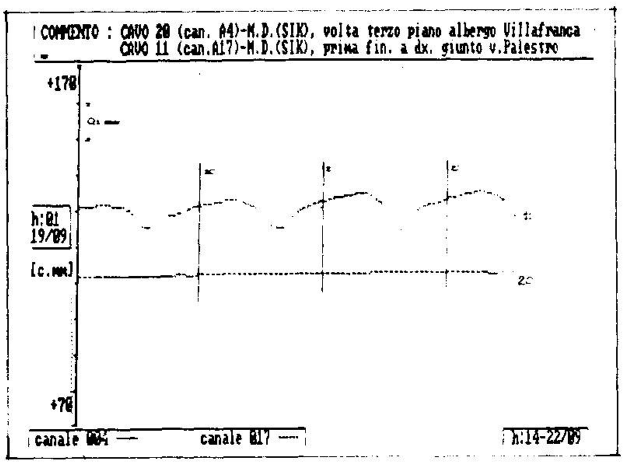
5.2.1 Modelling the structures

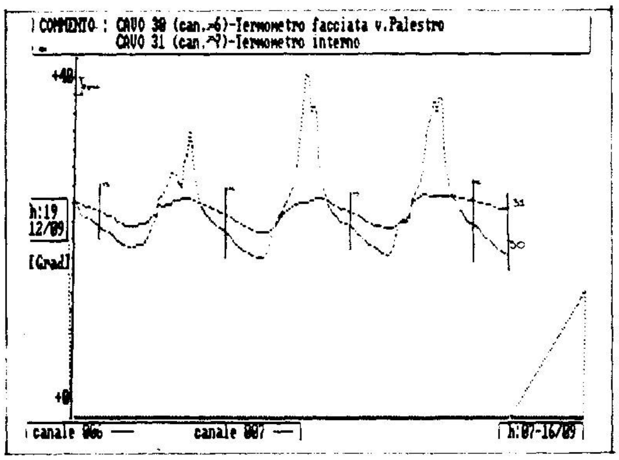
To analyse the stresses and the responses in the buildings studied, and especially in the areas where a well defined crack pattern existed and where the instruments were installed, finite element models were developed for the structures.

These were plane models for the faces of the buildings looking onto Via Palestro and Via Villafranca. The modelling was done using finite elements of the plate type. Rigidities corresponding to the thickness of the outside walls concerned were introduced into the models.

The voids (windows, doors, etc.) in the walls and structural discontinuities (joints and lesions) in the walls were also introduced into the models. Pairs of reference nodes were positioned at these discontinuities and their relative movement made it possible to measure opening and closing of the openings over the course of the structure's response to the disturbances induced by the tunnelling.

The structural models analysed extend below the road surface level, at which point finite elements were inserted whose rigidities simulate the behaviour of the foundation wells and of the surrounding ground. Figure 18 shows the mesh design for one of the models.





Figures 16 and 17 Two examples of the graphs produced

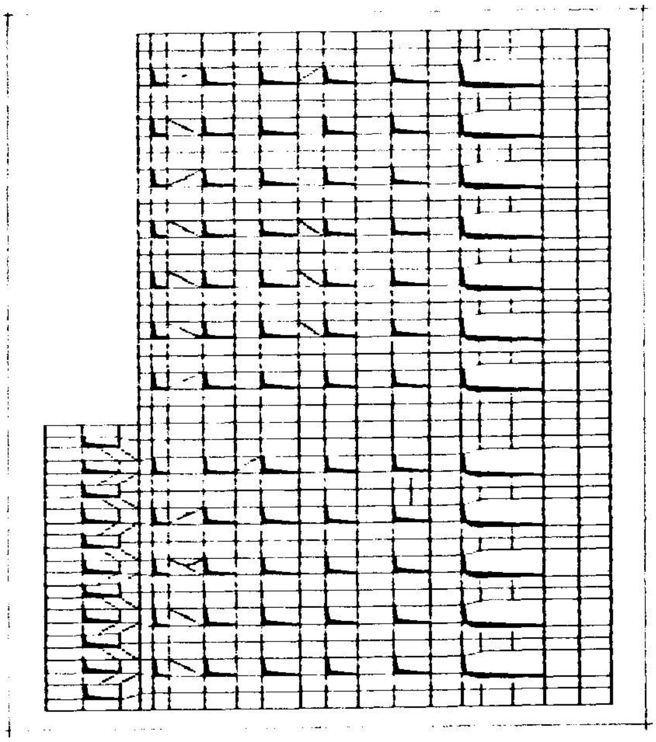
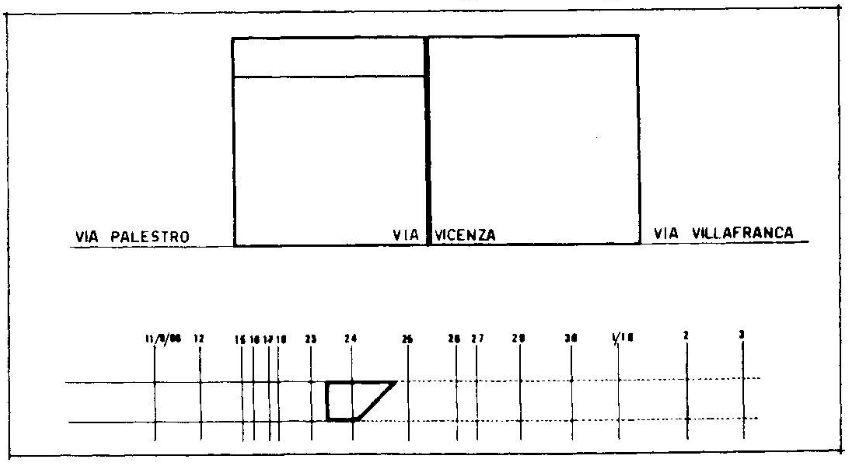


Fig. 18 Plot of mesh for one of the finite element models

Fig. 19 Progress of excavations



5.2.2 Analysis of the structural models

A series of static analyses in a linear elastic field was carried out on the models described above, applying imposed deformations to the base of the structures which corresponded to the values for subsidence recorded by level measurements during excavation.

Figures 20 and 21 show the subsidence diagrams measured at the base of the buildings on the Via Palestro and Via Villafranca faces respectively. These diagrams were drawn during the course of the excavations, the progress of which is shown in Figure 19 .

The analyses were carried out at different times, on each occasion using the subsidence values recorded in a generic time t_i as input values, and with different values for the mechanical characteristics of the materials making up the masonry structures and ground.

This made it possible to calculate the theoretical response of the structures over time and compare it with the data recorded by the instruments. It also made it possible to carry out sensitivity analyses on structural behaviour in relation to both uncertainties affecting the mechanical characteristics of the materials and deterioration of these which might locally alter the behaviour of the structural elements. The results of these analyses are discussed below.

5.3 Correspondence between theoretical analysis and data measured - Interpretation and forecasting of structural behaviour

5.3.1 Ground deformation and behaviour of lesions and joints. Evolution of the phenomena over time

In general the results of the analyses carried out on the theoretical models show good agreement with the measured data (see Figures 22-24). The comparison was made at different times during the course of excavation and showed that it was possible to obtain a clear correlation between the real behaviour of the structure and theoretical predictions.

Figures 25-29 show the extent and variation over time of the subsidence together with the measurement at the base of the buildings, movements of the lesions and of the joints. Figures 25 and 26 are for the Via Palestro face and show changes in subsidence over time with the corresponding changes in the opening of the joint between the two buildings examined and of the lesion measured with crack meter No. 12 (see Fig. 22).

The same Figures 25 and 26 show that most (about 60%) of the rotation about the joint occurred in the period between 12th and 17th September 1986, the day on which the shield reached the left hand building for about a quarter of its length. The remaining 40% of the rotation took place in the period from 18th September ti 1st October at an almost constant rate, if one excepts a slight temporary increase on the 25th and 26th.

Immediately after the beginning of October, when the subsidence had stopped, (as had the changes in the instrument readings), level recording on Via Palestro was terminated. At this time the shield was crossing Via Villafranca at the opposite side of the block.

The behaviour of the joint over time seems to be in good agreement with the changes in subsidence over time and also with the behaviour of the lesion measured with crack meter No. 12.

Figures 27 and 28 show the relationships between the changes over time in the subsidence measured at the data points on the Via Vicenza face and the bahaviour of the joint between the two buildings on the same face. The following points deserve specific mention. First, the slightness of the subsidence and joint movements recorded up to 17th September (when the shield was entering below the building to the left of the joint) and, second, the

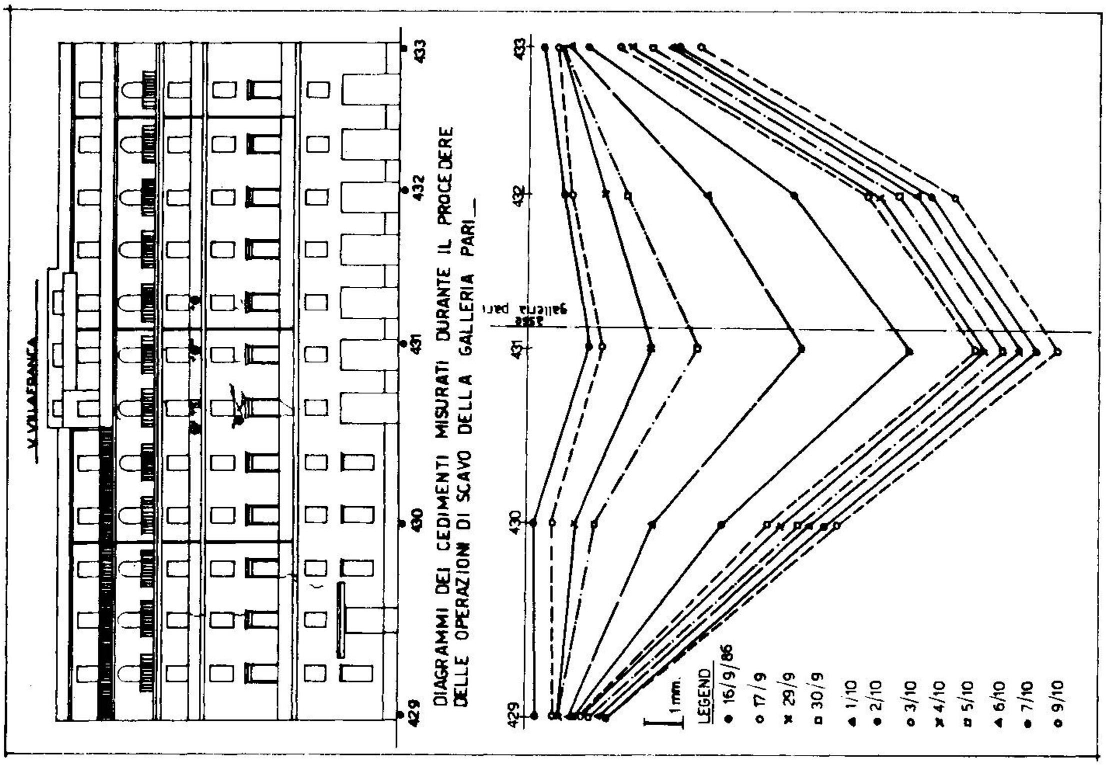


Fig. 20 Settlements diagram measured at the base of the building on Via Palestro

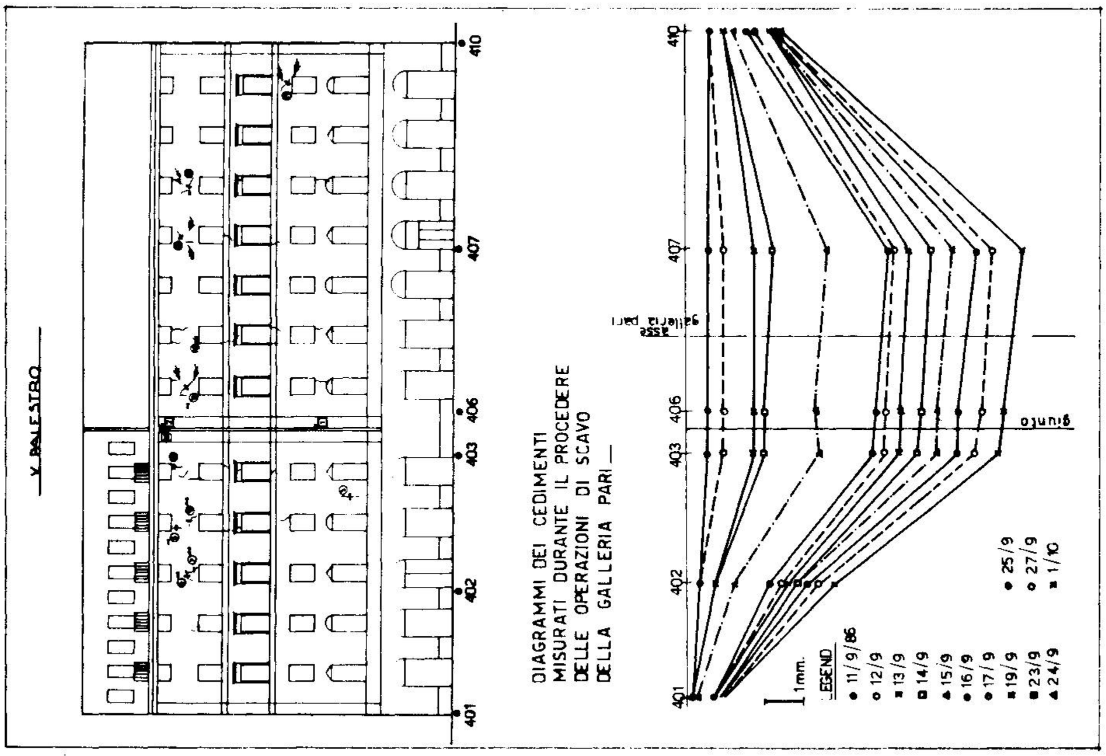
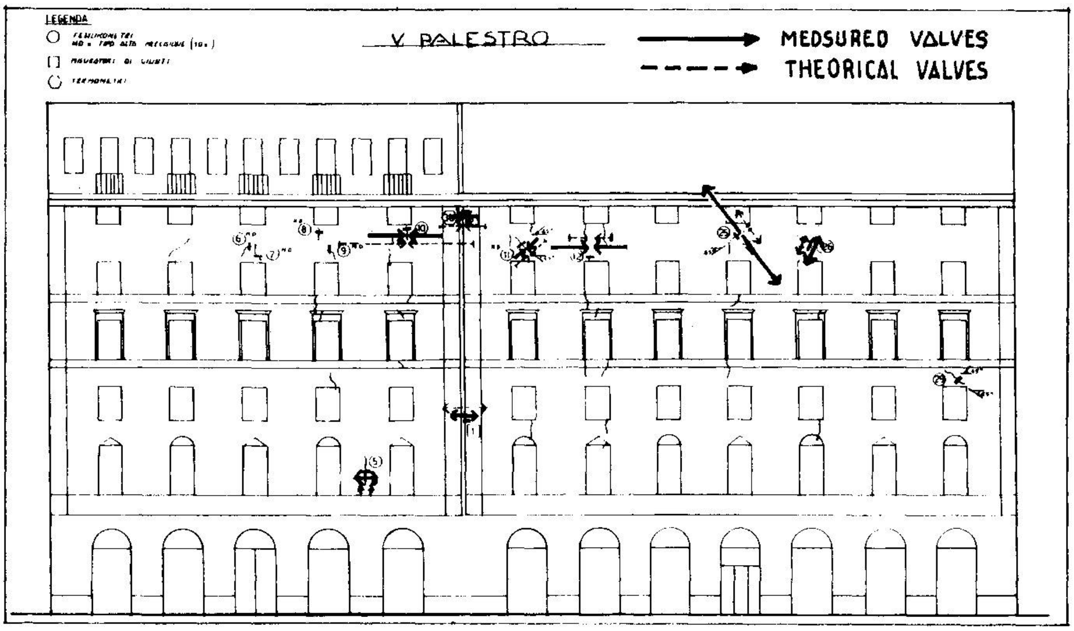
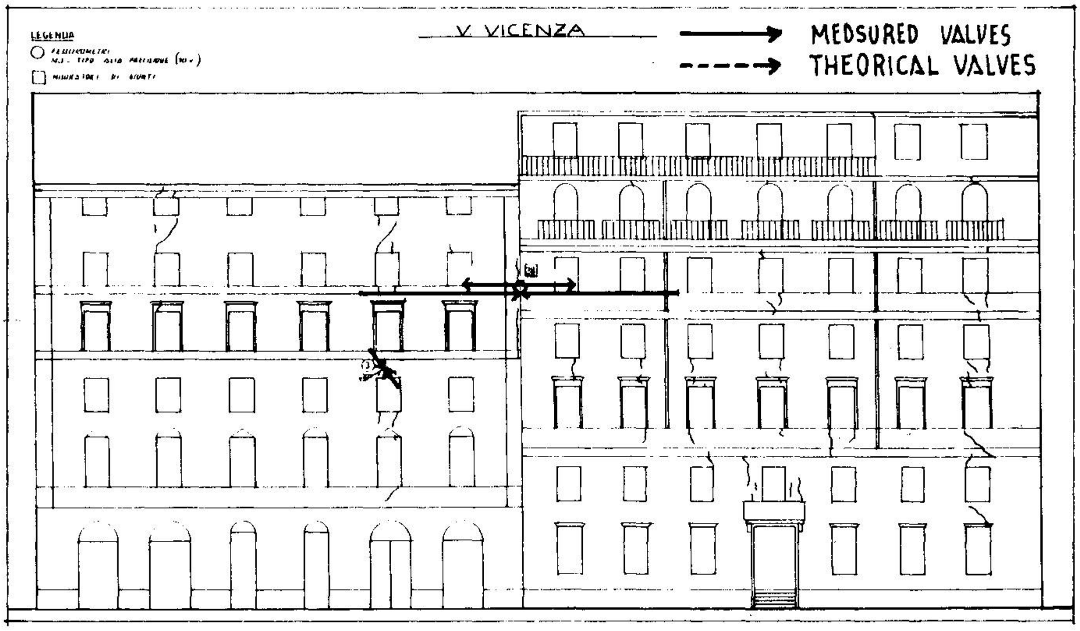


Fig. 21 Settlements diagram measured at the base of the building on Via Villafranca





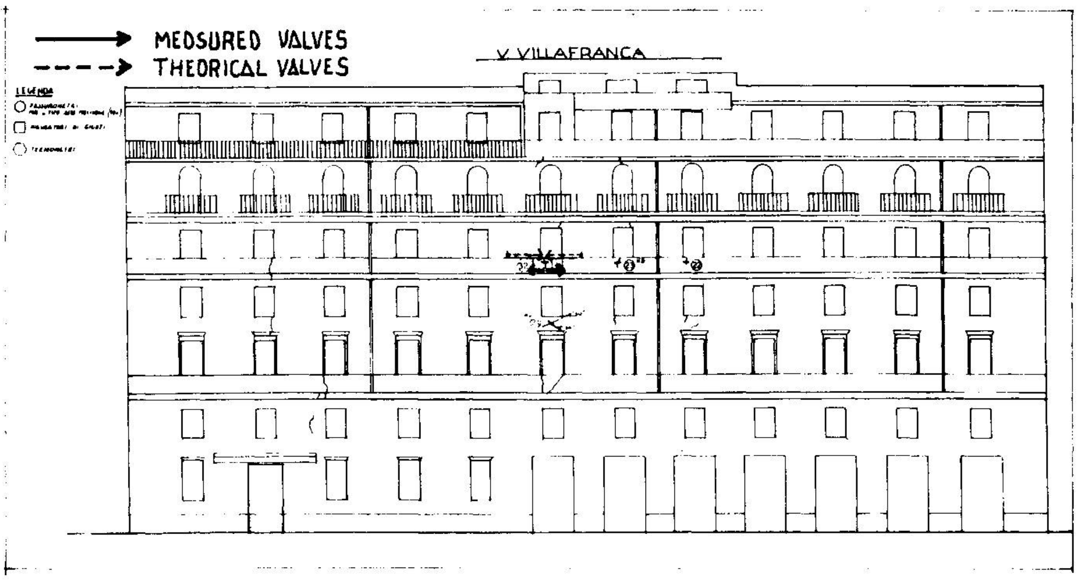


Fig. 22, 23 and 24 Corrispondence between theoretical analysis and data measured

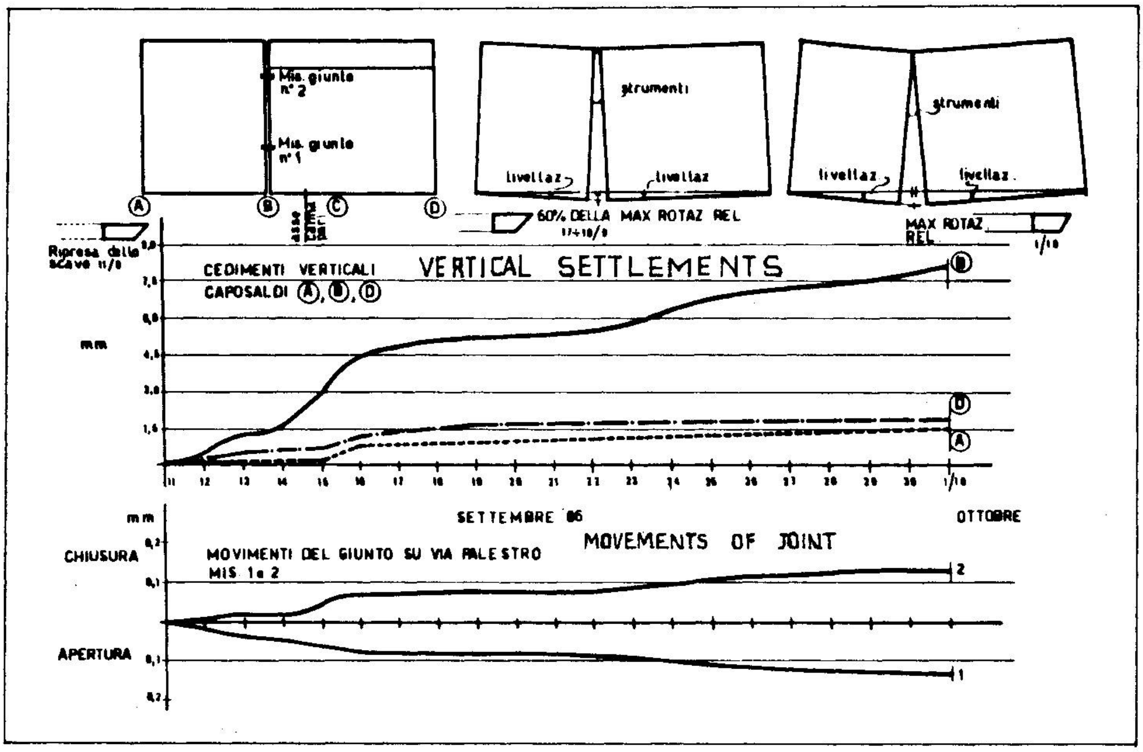


Fig. 25 Comparison between the measured settlements at the base of the building on Via Palestro and the movement of the joint.

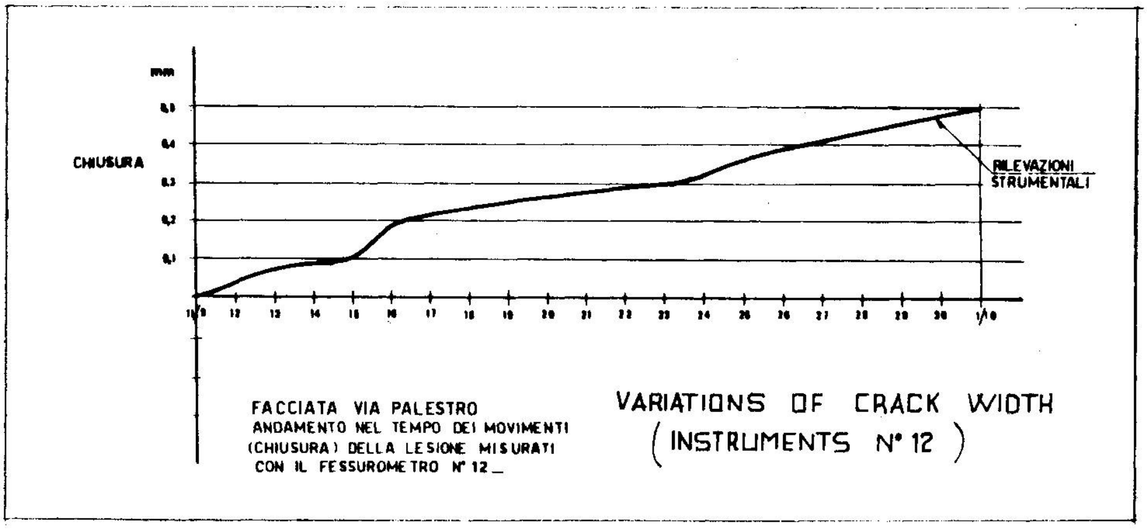


Fig. 26 Facade looking on Via Palestro
The variation over the time of the movements of the lesion on which was installed the fissurometer n° 12

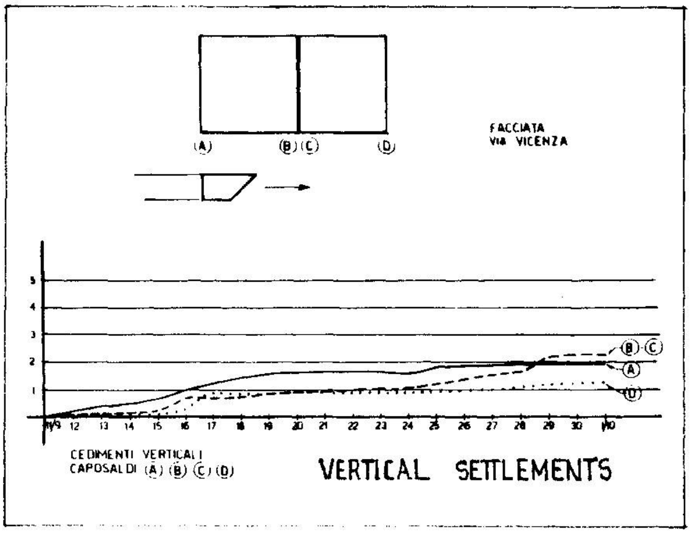


Fig. 27 Measured settlements on the base of the building on Via Vicenza

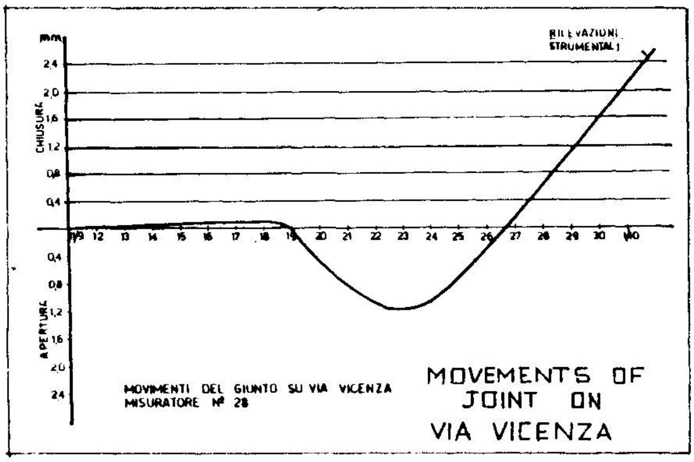


Fig. 28 Movements of the joint between the buildings on Via Vicenza

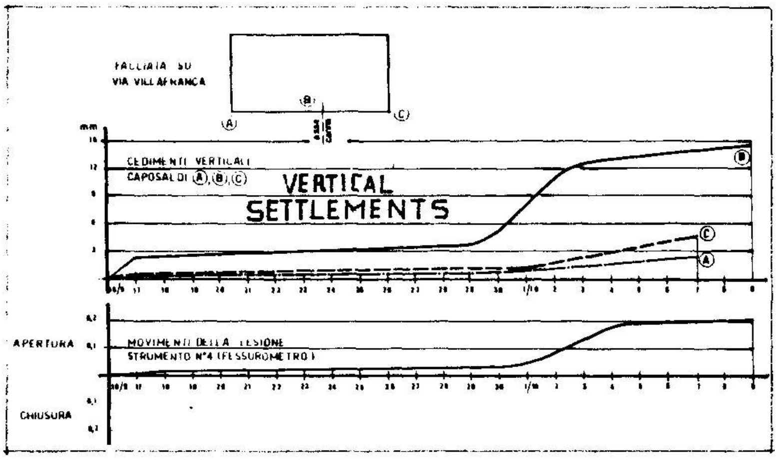


Fig. 29 Facade looking on Via Villafranca
Comparison between measured settlements and movements of the joint

inversion of rotation about the joint on inversion of relative subsidence between the joint area and the opposite extremity of the building towards Via Villafranca. This inversion took place about 29th September when the shield was approaching Via Villafranca.

From Figure 27 one can obtain a final value for the relative subsidence between datum points A and B (corner with Via Palestro, joint) of about 0.4 mm, with a corresponding figure of about 0.1 mm between points C and D (joint, Via Villafranca corner).

The relative rotation of the joint (the sum of the rotations of the two buildings, 24 and 26 m long respectively, is:

$$\varphi_{0\ tot.} = \frac{0.0004}{24} + \frac{0.001}{26} = 5.513 \times 10^{-5}$$

To this rotation there corresponds a closing of the upper end of the joint, where the sensor is fixed, of

$$5.513 \times 10^{-5} \times 16.5 = 0.9 \text{ mm}$$

relative to the bottom end, the height being about 16.5 m. The value actually recorded by the instrument was 1.33 mm. The difference between these two correspond to the deformation of the structures.

Analysis of an elastic model based on F.E. of the two buildings for the above subsidence gives a value closing of the joint of about 0.12 mm. The total closing is therefore:

$$0.9 \times 0.12 = 1.02 \text{ mm}.$$

The higher value measured experimentally can be attributed to the greater deformability of the structures resulting from the crack pattern described previously.

Finally, Figure 29 shows the correlation between the subsidence of the datum points on the Via Villafranca face and the behaviour of the lesion instrumented with crack meter No. 4.

5.3.2 Ground deformations and stress induced in the structures

As already said, the analyses carried out on the theoretical models of the buildings provided the stresses induced in the masonry structures as external conditions varied, that is, as subsidence as the base resulted from excavation. In general, there are two principal features of the results.

The first is the good agreement between the distribution of the increased stresses induced by subsidence and the behaviour of the joints and lesions. The second is the low level of increased stresses induced in the materials (normal stresses of some Kg/cm^2 and tangential stresses below 0.5 Kg/cm^2).

This provides a theoretical counterpart for what was measured directly during the course of the excavation, that is, the absence of phenomena of significance in terms of the safety of the buildings.

6. CONCLUSIONS

The measurements made on the structures of the buildings in question were intended to make a contribution to the study of problems with the excavation of tunnels in urban areas, in particular the static safety of masonry structures on the surface.

The studies carried out showed that the instrument network developed functioned reliably and correctly and provided an accurate picture of the evolution of the deformations in the structures, though in the absence of phenomena of great significance.

The instrumentation installed also made it possible to continue with all the normal activities carried out in the buildings (hotels as well as dwellings), ensuring the safety of the occupants through real time observation of the behaviour of the buldings.

In the event of dangerous changes in the static picture, the instrumentation would have provided alarm signals in sufficient time for safety measures to be put into effect (shoring up, strngthening, evacuation etc.).

In the specific case examined, and thanks to the measures taken by the tunnelling contractor, prior to the start of excavation, to improve the mechanical characteristics of the ground concerned, the phenomena recorded by the instruments were well within safety limits. No critical situations occurred.

The instrument data were compared with the results of theoretical analyses of mathematical models of the structures. This made a full interpretation of the behaviour of the structures possible.

The results obtained show the particular importance of construction joints in masonry buildings subject to subsidence of the foundation soils. As has already been described, in the case of the buildings on Via Vicenza, thanks to the good functioning of the joint only about 30% of the deformation produced by the relative subsidence of the foundations went towards inducing a state of stress in the structures. The remainder of the deformation was transformed into rigid rotations of the two buildings around the joint.

This behaviour indicates the importance of the joint, which should not be rigid but allow for movement and rotation, with sufficient being left between adjacent buildings.

The main contractor for the extension to Rome Underground Line "B" is the "Intermetro S.p.A." company with Mr. Mario Cangiano as Director of Works.

The tunnelling contractor is "Girola". The instrumentation and checking of the buildings studied were supervised by Prof. Goirgio Croci, with the assistance of M. Biritognolo and S. De Vito.

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Monitoring of a post-tensioned Bridge during Demolition

Contrôle d'un pont a précontraite postérieure au cours de démolition

Überwachung des Abbruches einer Spannbetonbrücke

Idris PRICE
Partner
Gifford & Partners
Southampton, U.K.

A graduate of the Universities of Wales and London, Idris Price was involved in bridge research at the Transport and Road Research Laboratory, U.K., for 16 years. Since 1978 he has been a Consulting Engineer with a special interest in bridge inspection, repair and rehabilitation.

Peter LINDSELL
Lecturer
University of Surrey
Surrey, U.K.

A graduate of the University of Oxford, Peter Lindsell worked on highway bridge design for 7 years. He has been involved in the demolition of prestressed structures since 1970 and is now responsible for a research team specializing in the instrumentation of concrete structures.

Susanné BUCHNER
Research Officer
University of Surrey
Surrey, U.K.

A graduate of the University of Surrey, Susi Buchner has been involved in research there for the last 4 years. Her work has involved the instrumentation of prestressed structures, particularly during demolition. She is developing methods for determining the in-situ state of stress in concrete structures.

SUMMARY
The deterioration of the precast concrete segments in a 25 year old post-tensioned bridge made it necessary to demolish and replace the superstructure. Debonding of the severed prestressing tendons was generally limited to three metres from the cut point. Consequently, excessive levels of prestress force remained in the deck sections over the intermediate piers as self-weight moments decreased. This action caused yielding of the longitudinal steel at the in-situ joints and non-linear behaviour of the structure. Strain gaugin of prestressing tendons during demolition indicated that long-term losses of prestress in the central span were in the range from 35-50%.

RESUME
La détérioration des segments de béton préfabrique d'un pont à précontrainte postérieure, qui a 25 ans, a rendu nécessaire la démolition et le remplacement de la superstructure. La perte d'adhérence des tendons coupés précontraints était limitée à trois mètres du point d'incision. Par conséquent, des niveau excessifs de la force précontrainte sont restés dans les sections du pont sur les piles intermédiaires comme les moments dus au poids propre diminuaient. Cette action a conduit l'acier longitudinal à céder aux joints de construction, et au comportement non linéaire de la construction. L'indication de la tension des tendons précontraints en cours de démolition à montré, que les pertes de précontrainte à long-terme dans la traveé centrale étaient entre 35-50%.

ZUSAMMENFASSUNG
Die Verschlechterung des Instandes der vorfabrizierten Betonsegmente einer 25-jährigen Spannbetonbrücke erforderte Abbruch und Wiederaufbau des Brückenträgers. Der Verbund der Spannglieder löste sich allgemein nur auf etwa drei Metern von der Einschnittstelle. Daraus resultierten übermässige Vorspannkräfte über den Zwischenstützen, da die Momente infolge Eigengewicht abnahmen. Dies führte zum Fliessen der Längsbewehrung in den Fugen und zu nichtlinearem Verhalten des Brückenüberbaues. Dehnungsmessungen an den Spanngliedern während der Abbrucharbeiten zeigten, dass die Langzeitverluste der Spannkraft im Mittelfeld 35 - 50% betragen haben.

1. INTRODUCTION

The Taf Fawr Bridge, built in 1964, carried the A470 trunk road over a tributary of the River Taff north of Merthyr Tydfil, South Wales. The bridge was approximately 30m above the river and had an overall length of some 144m, in three continuous spans of 39m, 66m, and 39m. Designed as a segmental post-tensioned structure, the deck was assembled as a balanced cantilever. Full details of the original design and construction procedure were described by Lundgren and Hansen [1].

The segments and joints were numbered consecutively from the west end (Fig. 1). Each segment comprised four, 3m long, precast I-sections joined by in-situ top and bottom slabs to form a three cell box structure. The precast I-sections were temporarily held in place during erection by a diagonal Macalloy bar passing through the webs at the joints. The Macalloy bars remained in position in the permanent structure and were fully grouted. Longitudinal post-tensioning was then applied to the precast units so that two 19 wire/25mm diameter strands were anchored at each cantilever end, running through the previously assembled I-sections. The continuously reinforced slabs were cast and further post-tensioning was applied.

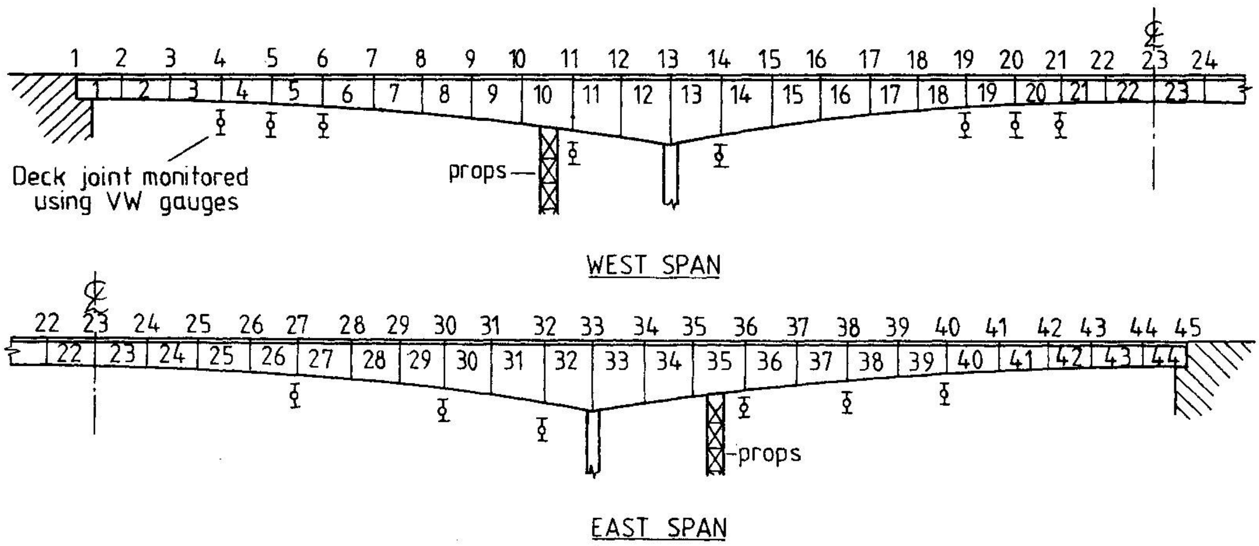

Fig. 1 Deck segment and joint numbering sequence

Tendons were provided in the top of the deck throughout the length of the superstructure to counteract the effects of moments induced during erection and under working conditions. Further tendons were added in the bottom slab, in the centre span and near the abutments, to resist the sagging moments after the bridge was made continuous.

In 1985 the decision was made to demolish the superstructure due to general deterioration of the outer precast sections. This deterioration enabled water and de-icing salts to leak through into the prestressing ducts causing corrosion of some of the tendons. The reinforced concrete piers were still in good condition and it was economically feasible to replace the deck alone. In order to remove the deck with the least damage to the piers, the dismantling process demanded a reversal of the construction sequence. Some operations were not wholly reversible, such as the reduction of prestress in step with the relief of dead load stresses. In addition, the behaviour during demolition was not entirely predictable since the extent of debonding on cutting the tendons was unknown. A number of monitoring procedures were utilised to minimise the uncertainty and to ensure that demolition was carried out in a safe and controlled manner.

The monitoring work was extended to include an assessment of the levels of stress remaining in the superstructure. Long-term losses of prestress, from tests on previous post-tensioned structures, have generally exceeded the original design predictions [2-4] and new techniques are being developed for determining the residual levels of prestress [5].

2. DEMOLITION SEQUENCE

Initially the superstructure was divided into two parts by cutting transversly at midspan. The central box units were then isolated by removing the outer I-sections. After providing temporary props under each side span, the connections between the superstructure and the abutments were severed. The box units were then lowered into the valley, maintaining stability by ensuring excess weight remained on the side span. Work commenced on the west span side sections and progressed onto the east span. The lowering of the west span central units ran concurrently with the removal of the east side sections. Finally, the east span central units were removed using the lifting frames transferred from the west side.

Initial trials, using a diamond saw to cut the midspan position, proved ineffective as the cantilevers dropped trapping the blade. The demolition work proceeded using a combination of pneumatic jack hammers and a Montabert breaker. The breaker was used to isolate each unit prior to removal by crane or lifting frame.

3. INSTRUMENTATION

3.1 Structural monitoring

During the removal of the central units four principal features had to be considered for monitoring, the first two being of prime importance from a safety aspect.

- Cutting poorly grouted cables could lead to excessive debonding of the steel occurring over several segments from the cut position. This could result in either ejection of an anchorage or problems in the demolition process. Only two cables connected some of the units to the remaining cantilever which had to support both the lifting frame and the end unit being demolished. Initial trials were carried out in the top slab of the north edge section of segment 14, the extent of debonding being assessed by monitoring the change in concrete surface strains which occurred along the line of the severed cables. Further debonding checks were carried out in the central units of segments 26 and 27.

- Excessive compressive stresses might develop in the top flange and tensile strains might be induced in the bottom flanges of the units near the piers. This effect would be due to unreleased prestress in the top slab, which could exceed the cantilever moments as self-weight was reduced.

- The pier reactions and rotations had to be monitored in order to ensure that the concrete hinge at the base did not crack as the weight of the deck was removed.

- In conjunction with the monitoring of the pier the general stability of the cantilever had to be assessed as each unit was removed.

In order to detect the first possible form of failure, vibrating wire (VW) strain gauges were attached to the inside of the box units across the in-situ joints at the ends of both the midspan and side span cantilevers. A sudden loss of prestress would be immediately apparent at the joints if tensile stresses began to develop at the top of the web. The second form of failure would produce tensile stresses across the joints in the bottom flange adjacent to the piers. Thus VW gauges were placed in these positions and adjacent to the temporary props. The information gained on the west side resulted in a revised gauging pattern for instrumenting the east span (Fig.1).

All structural monitoring of the superstructure was carried out remotely so that the demolition programme was unaffected and the risks to the monitoring team and the Resident Engineer's staff were reduced. Readings from the strain gauges and temperature sensors were taken from the adjacent temporary Bailey bridge. Changes were monitored on removal of each central unit and after movement of the lifting frames.

The last two monitoring checks could be carried out by instrumenting the concrete piers and steel props. The large area of the piers meant that little strain change would be noted and the readings might be swamped with temperature effects, however, two 140mm VW strain gauges were placed on either side of the pier close to the outer faces. In addition, two 65mm VW gauges were attached to each of the four props, on the neutral axis, to monitor the change in load due to the removal of each central unit. A number of the strain gauges applied also had temperature sensors so that corrections could be made for changes in the weather.

3.2 Research monitoring

Although the monitoring work was primarily confined to the removal of the central box units, checks were carried out on the level of residual prestress in the strands and the concrete stresses in the webs and flanges of the box. Three methods were used to assess the levels of residual stress in the superstructure:

- The prestress remaining in selected tendons was determined from measurements of the change in strain which occurred when a few wires were cut. Short lengths of strand were exposed so that a Demec gauge could be used to measure the released strains. Although complementary laboratory tests are required to interpret the strains measured, this appears to be the most direct and reliable method for determining residual prestress levels.

- An indirect method, involving the instrumentation of concrete cores, taken from the webs and flanges of units 25 and 27, was used to help in the assessment of the in-situ concrete stresses at sections where high shear stresses had caused severe cracking of the webs. Analysis of the data, obtained by taking strain measurements on and around the cores, relied upon a theoretical analysis of the effects of a hole introduced into a stressed infinite plate [6]. As the strains released around and across the cores relate to the residual stresses in the section, this method produces a summation of the effects of the prestress in the strands and the local stresses due to self-weight.

- The demolition of the structure itself provided another means of determining the concrete stresses. By instrumenting a complete unit and recording the total release of strain, it was possible to relate the stresses released at a section to the cantilever moment due to self-weight and residual prestress.

4. RESULTS

4.1 Structural monitoring

4.1.1 Bonding

The north edge section of segment 14 was selected for initial debonding trials and the cables were exposed at either end of the unit so that the strands could be cut in increments. This also provided an opportunity to measure directly the level of prestress in these strands.

Generally the trials showed that fully grouted tendons would not debond more than a fraction of the length of a unit, ensuring that virtually all the residual prestress force would be present at the next in-situ joint. During progressive cutting of a tendon, the changes in strain on the adjacent concrete surface illustrated the debonding characteristics (Fig.2).

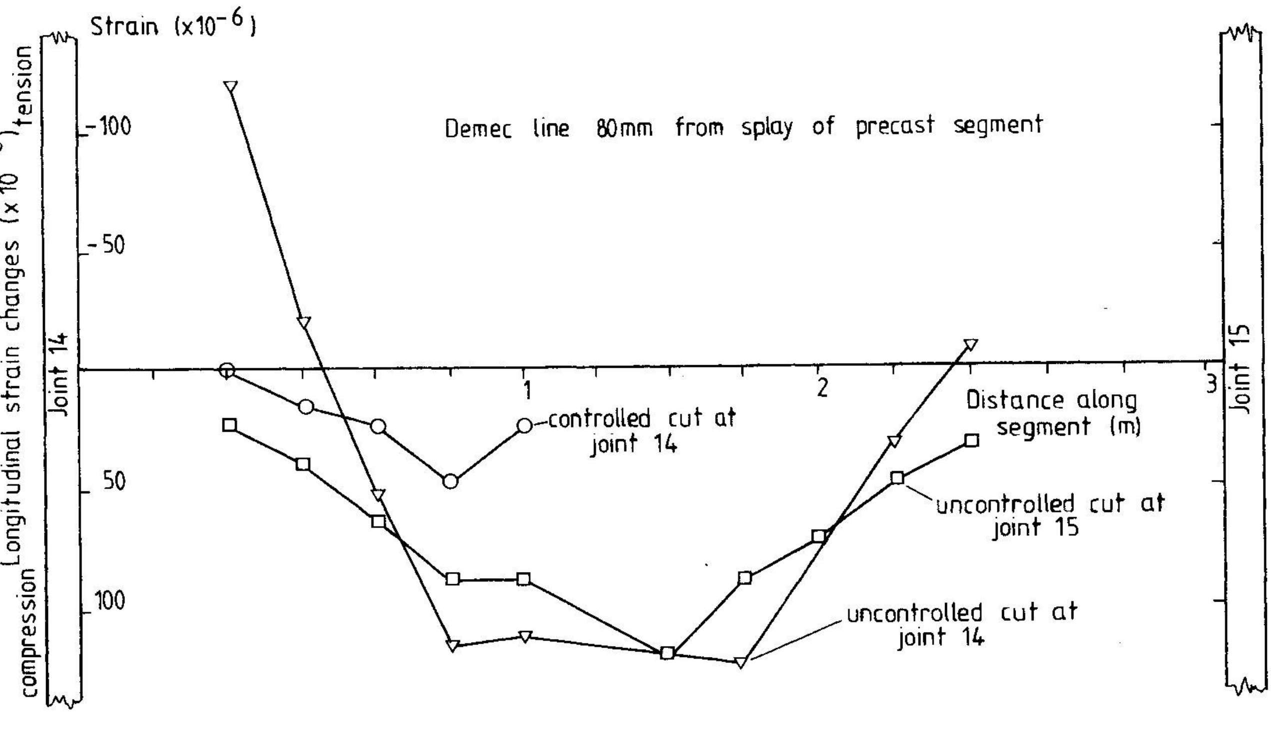

Fig.2 Typical debonding curve.

The curve for the controlled cut at point 1 relates to the cutting of 3 wires in the nearest 19 wire strand at joint 14. The curves for uncontrolled cutting at points 2 and 3 relate to the severing of all the tendons at joints 15 and 14 respectively. As the cutting at point 3 (joint 14) was carried out last, the corresponding curve shows a symmetrical distribution in the strain changes. Each half of the curve is of a form associated with the transfer of stress in the transmission zone of a pretensioned tendon. High strain gradients near the cut end of the tendon are a consequence of the transfer of force from the steel to the concrete by bond. The transmission length is of the order of a metre. In the absence of bond, a fairly uniform tensile strain change would probably be experienced, relating to the release of the general prestress in the segment.

Further checks on the debonding were made in the central units 26 and 27: the results confirmed the original findings. It was concluded that for 60% of the tendons, prestress would be re-established well within the length of a unit. This meant that there was likely to be a substantial increase in the stresses over the piers which could be monitored by the VW strain gauges positioned across the nearest joints.

4.1.2 Behaviour of joints

Generally the strain changes due to the removal of the first box units were relatively small. The readings taken across the joints furthest from the piers were closely predicted by elastic theory. However, as demolition progressed the strain changes in the joints near the piers increased dramatically. This effect was particularly noticeable for joints 14 and 32 on the centre span. Although joint 32 did not initially suffer significant strain changes, after the first five midspan units had been removed the apparent tensile strains increased to become similar to the readings from joint 14 (Fig.3).

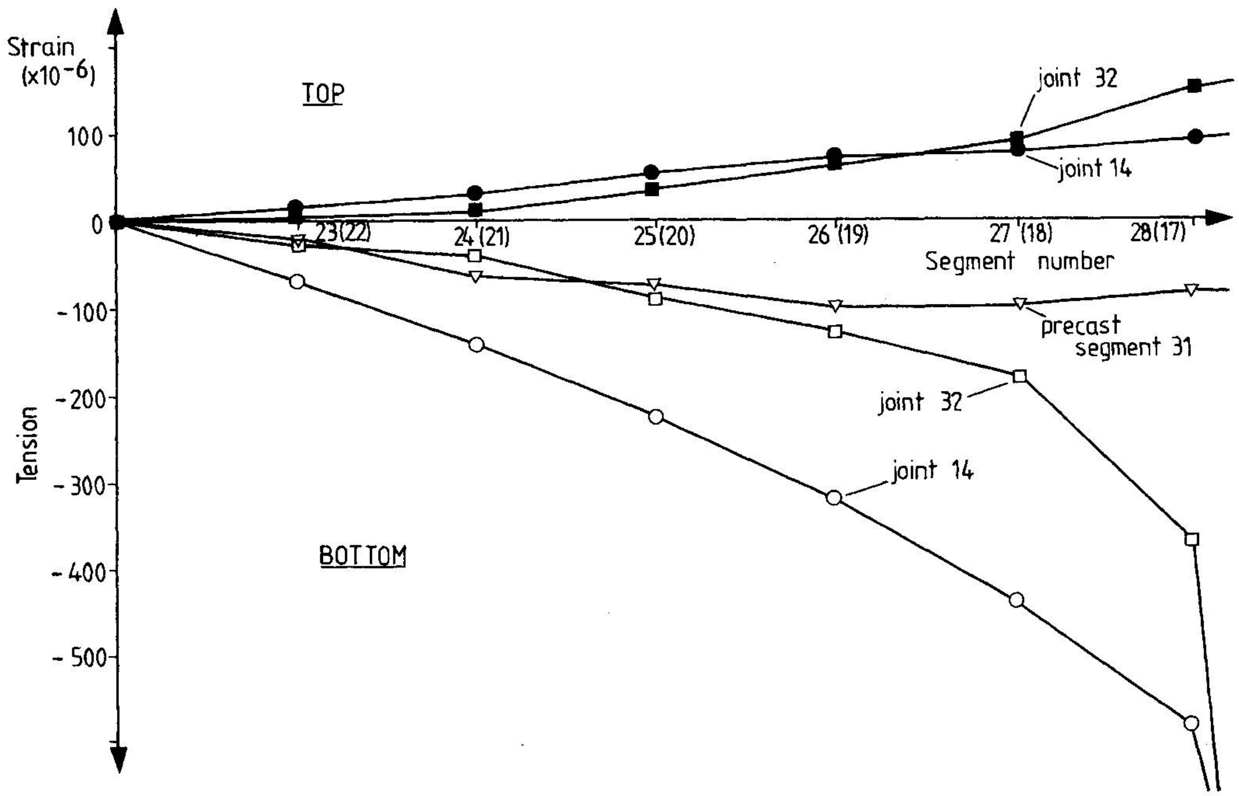

Fig. 3 Comparison of strain changes on south side.

As joint 32 began to show signs of cracking the strain gauge in the adjacent unit No.31 recorded no further increase in strain. The unit could no longer transmit forces below the neutral axis, except by yielding of the mild steel bars in the bottom slab and the two diagonal Macalloy bars that passed between the segments. It appeared that the bottom flange steel would yield well before the demolition sequence was completed and that conventional prestressed concrete theory could not be used to predict the stresses remaining in the cantilever sections as the joints did not retain the full second moment of area associated with the precast units.

4.1.3 Stability checks on piers and props

As the piers were such massive structures, temperature variations were likely to swamp the small strain changes caused by removing the central units. Hence the need to monitor the gauges on the steel props where the strain variations were relatively large. The strain changes were converted into equivalent loads on the falsework and hence the reactions on the piers could be estimated. Constant monitoring of the props showed that no unusual effects occurred during the demolition process.

It was possible to determine the effective Young's modulus for the concrete pier by monitoring the effects, of removing a unit, on the pier and prop. Unit 17 on the midspan side of the pier was chosen and, although the pier strain readings were subjected to the errors caused by the low magnitude, a realistic value of 31.4 kN/mm^2 was obtained for the modulus. A similar exercise was attempted during the removal of unit 39 on the east side span. Unfortunately temperature effects completely swamped the extremely small strain changes in the pier.

4.2 Research monitoring

4.2.1 Direct measurements

Direct measurements of the residual strain in the prestressing strands indicated losses ranging from 35-50% for both units 25 and 26. These results are comparable with measurements taken from other post-tensioned segmental structures of this age [2-4]. The bottom slab cable in unit 25 indicated losses of 35% which showed that debonding losses due to the initial cut some 8m away at midspan had not occurred.

Measurements in segment 14 produced the highest residual steel strains recorded and suggested losses of only 26%. More measurements were needed in the vicinity of the supports to confirm whether the losses were generally lower in this region.

4.2.2 Indirect measurements

The coring method indicated a general pattern of consistency, although there was a degree of unexpected behaviour. In particular, the high shear stresses in the webs caused unusual effects across and around the cored holes because of the high tensile stresses in the shear steel. Only the strain changes on the concrete cores could be considered to give a guide to the stresses in the section. The major principal stresses from the cores in the north web of unit 25 appeared to lie in the range from 2.3-2.7 N/mm^2 at an angle of 58^o to the longitudinal axis. The stresses from the south web were higher at 3.3-4.2 N/mm^2. There was a general degree of consistency in the angle of the major compressive stress of approximately 55^o to the horizontal.

The results from the coring trials carried out through the top slab of unit 27 and the bottom slab of unit 25 showed that these areas were under complex stress conditions. No definite patterns could be ascertained for the various gauging positions as there was very little consistency among the readings. However, in the top slab of unit 27 there were indications from the core results that the longitudinal compressive stresses ranged from 2.0-3.0 N/mm^2. In the bottom slab of unit 25 both core readings indicated longitudinal stresses of 2.5-4.0 N/mm^2.

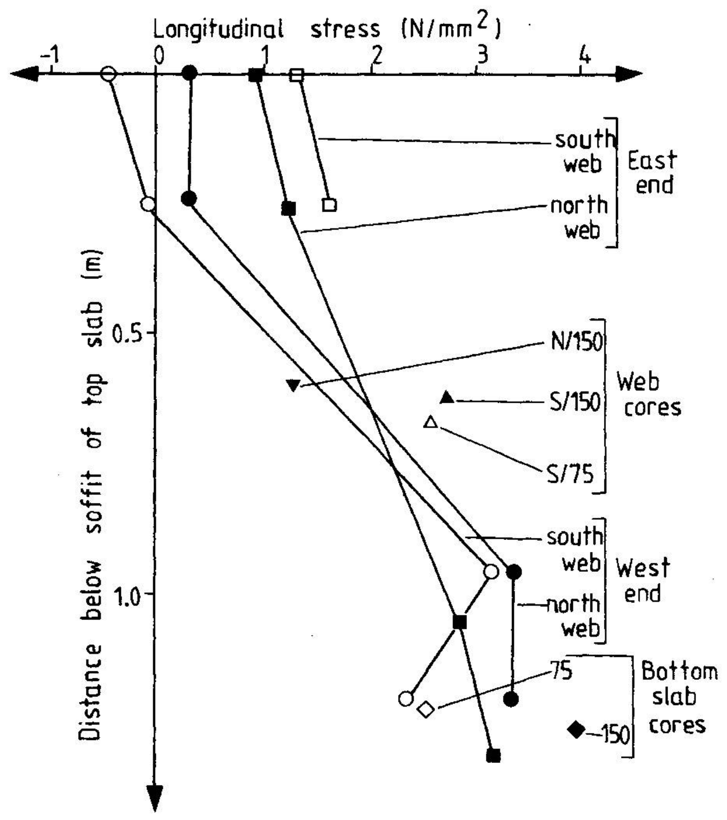

Fig. 4 Longitudinal stresses in unit 25

The stresses derived from the coring method were compared with those obtained from the complete removal of unit 25 (Fig.4). These longitudinal stresses were in reasonable agreement for both the north and south webs at each section although, as expected, there was a distinct difference between the two sections. Despite the fact that the conditions were not identical for the two trials, the results were in good agreement. This indicated that the coring method provided useful indirect evidence on the state of stress in the webs and flanges.

5. CONCLUSIONS

The demolition of Taf Fawr Bridge provided the opportunity of applying various monitoring techniques. The results contributed not only to the development of these methods but also to the safety of many of the operations. It was shown that when a well grouted tendon was severed, the prestress was probably re-established in the concrete within 1m of the cut and certainly well within the 3m length of a unit. For this reason,and because most tendons were found to be well grouted, the units on the central span could be dismantled with an adequate factor of safety.

The total loads on the temporary props, caused by the removal of these units, was closely monitored and corresponded with the reactions anticipated by the contractor. As the demolition progressed and the level of prestress increased in the remaining units near the piers, cracking developed at the joints in the bottom flanges and the strain distribution became non-linear. Warning of the impending yield of the longitudinal reinforcement, in the bottom of the in-situ concrete at the joint, was provided by the VW gauges, but the residual resistance of the section in flexure and shear could not be predicted.

Residual prestrain measured on the tendons indicated likely prestress losses in the range from 35-50%, although in the support region this may have been considerably lower at 26%.

An indirect method using 150mm and 75mm diameter cores can provide a basis for assessing the in-situ stresses in concrete structures with little damage. The results from the coring trials corresponded closely with the total release of stress which occurred on removing unit 25.

6. ACKNOWLEDGEMENTS

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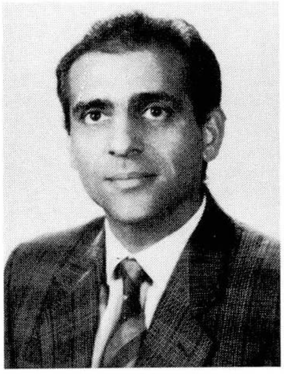
Testing and Monitoring Activities of Vega A Project

Essais et surveillance du système offshore de Vega A

Prüfungs- und Überwachungstätigkeiten im Vega A System

Ostilio SPADACCINI
Associate Professor
University of Florence
Florence, Italy



Ostilio Spadaccini, born 1946, obtained his Civil Engineering Degree from the University of Pisa. For ten years he was Technical Manager in a Steel Construction Company and is now professor of Steel Structures in the Civil Engineering Department of University of Florence and consultant for marine structures with SELM, Società Energia Montedison.

SUMMARY
The Vega Field offshore system, the most important on the italian shelf, includes a fixed drilling and production platform located in 123.0 m water depth, a sealine comprising three 2.3 km lines connected to a single point mooring structure for a 250,000 DWT tanker. Continuous and extensive testing and monitoring activities were performed during construction and installation; structural and cathodic monitoring systems were provided to collect usable data for the periodic inspections during the 25 year operating life.

RESUME
Le système offshore de Vega, le plus important du plateau italien, est composé d'une plateforme fixe de forage et de production, située à une position où la profondeur d'eau est 123 m, d'une sealine d'une longeur de 2,3 km ayant trois lignes différentes connectées à une structure du type monobouée pour amarrer un pétrolier de 250.000 DWT. Des activités élaborées d'essais et de surveillance furent effectuées pendant la construction et lors de l'installation; des systèmes de surveillance de la structure et de la protection cathodique furent installées, afin d'obtenir des données à utiliser lors de l'inspection périodique au cours de 25 années de vie opérationnelle.

ZUSAMMENFASSUNG
Das Vega Offshore System, das wichtigste des italienischen Plateaus, besteht aus einer 123 m wassertief stehenden Bohrungs- und Produktionsplattform und drei 2,3 km langen Förderleitungen, verbunden mit einer Mono-Boja für einen 250.000 DWT Tanker. Während Zusammenbau und Installation wurden fortgesetzte Prüfungs- und Ueberwachungstätigkeiten durchgeführt. Ueberwachungssysteme für Struktur und kathodischen Schutz dienen der Aufnahme von periodischen Daten zur Systeminspektion während der vorgesehenen 25 Betriebsjahre.

1. INTRODUCTION

Vega field, operated by SELM, Società Energia Montedison, contains more than 1 billion bbl of oil, of which about 30% may be recoverable. The crude has an API gravity of 15.5°, a pour point of 18°C and a viscosity of 1.000 c.p. when "alive" and warm. The crude properties dictate use of several special production and transport techniques involving light oil blending, heating and thermal insulation of sealine. Vega is located in Italian waters between the southeastern coast of Sicily and the island of Malta, about 15 miles offshore Ragusa. The field, which lies under a maximum water depth of about 131 m, is some 14 km long and varies in width from 1 to 2 km. The entire field area is highly faulted and fractured, which contributes significantly to producibility of the carbonate reservoir. The reservoir lies at subsea depths ranging from 2440 m to 2650 m. Productive thickness averages some 250 m and ranges to 350 m maximum [1].

Vega is a large enough structure to require at least two platforms for its development. The scheme selected for the first phase of the Vega oil field development includes a drilling and production platform, eight leg jacket type, 140 m high and weighing 11,000 tons, the sealine which comprise three different lines, two 10" thermally insulated lines and an 8" for diluent, a single point mooring system located 2.3 km from the platform and a storage tanker specially converted for service as floating storage offloading unit of 250,000 DWT. The system was designed for a life of 25 years with a design production flow of 60,000 BOPD.

Before starting the development of the field and drilling activity a site survey, soil sampling and geotechnical investigation were performed to provide information concerning the surface and subbottom conditions, and to determine the soil conditions related to installation of the gravity buoy and the platform with its pile foundations. Environmental loads (wind, wave and current) based on historic data were analyzed for the 100, 25 and 1 year storms. For 100 year storm the maximum wave was 18.3 m height, period of 13.0 sec. and length 273.8 m in the WNW direction, associated with a maximum instantaneous gust of 59.0 m/sec. The 25 year wave height distribution and associated wave periods were used in fatigue analysis. The 1 year return period storm current was assumed to act collinearly and simultaneously with all waves and was used to calculate the fatigue wave loading.

The effect of earthquakes on the structural behaviour of the platform were considered on the basis of a seismological and vibratory ground motion evaluation. This included modelling of seismicity and definition of seismogenic provinces, selection of attenuation functions, probabilistic analysis and vibratory ground motion.

2. MAJOR VEGA A COMPONENTS

The first stage of Vega field development was dependant upon the following factors:

- oil type and reservoir characteristics,
- water depth and environmental conditions (wind, sea, seismic),
- soil conditions and stratography,
- drilling of the wells during onshore platform construction to allow an early production,
- simultaneous drilling and production activities.

The adopted solution with a fixed platform, sealine and single point mooring allowed applicable operational requirements and safety aspects to be satisfied.

2.1 Platform

A subsea template with 18 well slots was fabricated and installed in September 1983 before starting drilling operations. This is a 37 m long structure, with four 36" grounted piles. Sufficient stiffness was provided to meet tolerance needs of future tieback connections between predrilled wells and fixed platform. Specifically an acceptance limit of 1 degree was set for the structure planarity. To meet this tolerance a jack levelling system was provided complete with : two electronic inclinometers installed with two indicators, one local,

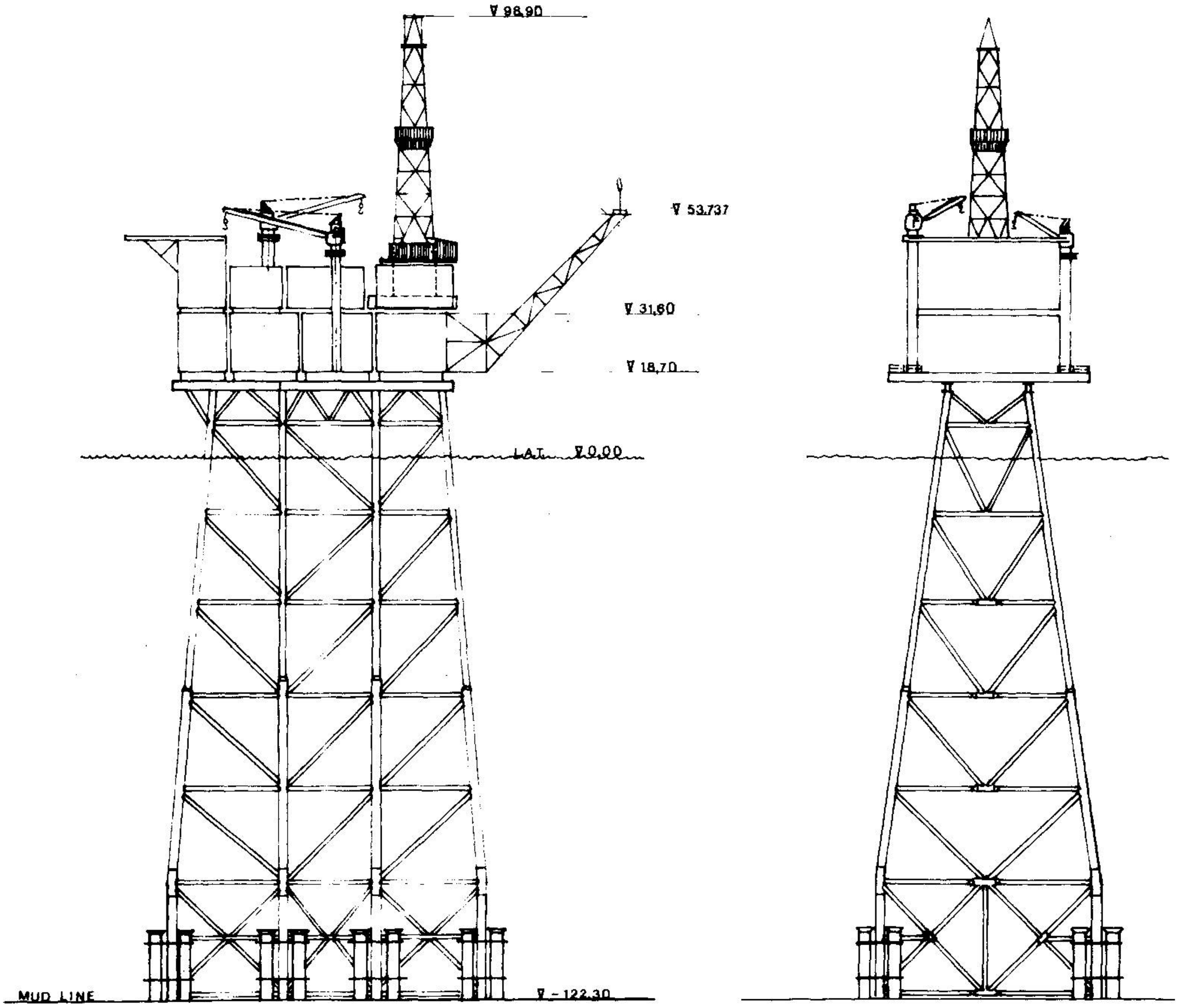


Figure 1 : North West and South West views of Vega A platform.

installed on the template, one remote installed on board installation barge; an optical slope indicator on the template as a back-up of the electronic system.

To avoid the interaction between soil and structure flexible type connections were provided between 30" conductor pipes and template so that a defined maximum load would be transmitted to the template.

The jacket is an eight leg tubular space frame structure, as indicated in figure 1, approximately 140 m high, with bottom dimensions of 68x58 m and top dimensions of 46x18 m. There are six horizontal frames spaced at roughly 24 m intervals, each of which supports the construction guide frame.

The platform deck is composed of two skid beam rows on which each production and drilling modules is placed and supported in four points. Modules have independent structures and are not connected to one another structurally to respect the design dynamic seismic behaviour. The platform is supported by 20 steel vertical piles driven to approximately 65 m below the sea bed. These piles were driven using an underwater hammer.

Auxiliary temporary buoyancy tanks were connected to the jacket for use during the launch and positioning on the template. The main advantage of this type of self-up-ending jacket is the reduction of time required for installation. However, on the other hand, this procedure has some impact on the installation safety because it is impossible to perform any kind of emergency action after jacket launching. These considerations required that the weight of the structure and the buoyancy distribution be continuously monitored throughout fabrication [2].

The maximum weight of a single module is about 1300 tons, the total operating weight of topside modules is about 13,300 tons. Two flexible appurtenances, flare and drilling rig, required a special dynamic analysis under seismic loads since the pseudo-static analysis used for design of module structures was not considered sufficiently conservative.

2.2 Single point mooring (SPM)

The SPM is an articulated column anchored to the sea bed by a gravity base. The storage tanker is permanently moored to this column by a rigid yoke [3], see figure 2.

The gravity base, 33 m square and 13 m high and weighing 750 tons supports at its centre the universal joint. Due to the poor soil characteristics 2 m deep skirts under the base drums and beams, see figure 2, are necessary for stability. The universal joint connects the base to the lower part of the column by self-lubrificating bushings which allow column articulation. A cylindrical steel column is the largest component of the mooring system with a dry weight of 2000 ton, a diameter of 9 m and 125 m length. The column contains heavy ballast materials in its lower part and tanks for buoyancy and stability in its upper part. At the top of the column a triaxial joint is installed which gives the yoke adequate degrees of freedom with respect to rotation, pitch or roll.

2.3 Sealine

The sealine comprises three separate lines , two 10" thermally insulated lines for oil service and one 8" uninsulated line for diluent. The 10" lines are a novel design comprising an insulated double pipe system and special forged connectors. This solution has the following advantages:

- mechanical protection of thermal insulation during laying and during the life of the system,

- in the case of damage to one section of the pipeline the effect is localized and does not significantly affect the general thermal efficiency,

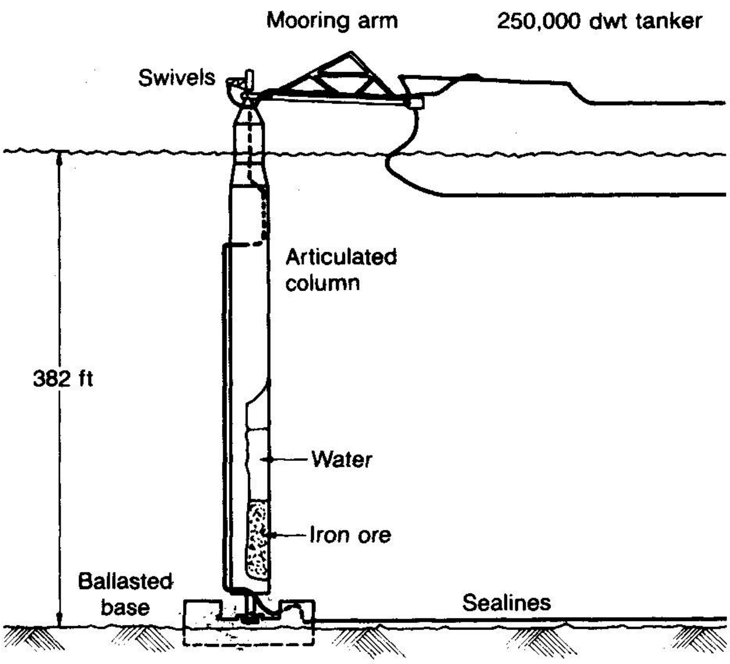


Figure 2 : Single point mooring for tanker at 2.3 km from platform.

- good stress distribution between internal and external pipes.

3. CONSTRUCTION AND INSTALLATION

The most important testing and monitoring aspects during the construction and installation phase were:
- steel material control and welding quality control,
- weight and dimensional control of the structures,
- control and monitoring of marine operations (i.e. launching, docking, ballasting and piling for the jacket; floating, docking and ballasting for the SPM; laying for the sealine).

3.1 Materials

Two type of high strength steels (type I and type II) were used for the primary structure, basically in compliance with EU 25 Fe 510 with some modifications. The main differences between type I and II steel are: the requirement for type II of through thickness tensile tests per EU 164-83, and the thickness for type I is less than 60 mm. The maximum carbon equivalent (C.E.) according to the formula $C + Mn/6 + (C_r+Mo+V)/5 + (Cu+Ni)/15$ was 0.42% in ladle and 0.44% in the product, the soluble aluminium to nitrogen ratio to be at least 2:1. Impact test was in accordance with EU 45 with Charpy V-notch, specimen being removed transverse to the rolling direction and the minimum single value 27 J for thickness $\leqslant$35 mm at -20°C and for thickness $>$35 mm at -40°C.

The qualification tests for the base material were the following: tensile, bend, impact, ageing, drop weight, COD, macroetch. The qualification tests for the weldability were: cold short cracking test, impact in HAZ, hardness and macroetch of joints, COD in HAZ.

During the workshop construction of jacket nodes, some times during ultrasonic testing defects appeared in the welding especially for the type I steel, which at the next repair control were found to originate in the base material.

In the yard during the assembling of nodes with members by single side butt welds defects were originally found due to lack of accurate fit-up, welder not sufficiently skilled for the first pass of single butt weld, incorrect interpretation of UT results.

3.2 Weight and Dimensional Control

Continuous monitoring of the weight and dimensions during fabrication is essential for marine operations engineering.

For the jacket the weight and centre of gravity was monitored and the as built situation was used for final check of load-out, transport and launching.

One of main problems during the construction was the control of node geometry, however the tolerances achieved for the jacket were:
- the horizontal distance between the centre lines of adjacent columns at each horizontal frame within 6 mm,
- at each horizontal frame the diagonal distances measured at column centerlines not differ by more than 15 mm,
- jacket conductor guide centerlines are not deviate more than 12 mm from construction drawing.

3.3 Marine Operations

The marine installations of the jacket, S.P.M. and sealine included various monitoring activities during the launch phase, docking, ballasting and piling.

3.3.1 Jacket

During positioning, the jacket was continuously monitored using an acoustic system. This system, linked to crane barge with a cable and acustic hydrophone, comprised a microprocessor based control and telemetry unit (CTU) with internal sensors for roll, pitch and depth, and four external sensor transponder/remote transducer (TRT) units. The four TRT's were mounted at -120.3 m level on the inside face of legs. Four compact transponders were pre-installed on the template corners.

A control unit installed on the crane barge was used to request the CTU to select a master transponder and cause it to interrogate one of the four template transponders.

The above measurements were up-dated every 12 seconds either in hardware or acoustic telemetry.

A secondary jacket positioning system was provided utilizing underwater cameras and lights mounted on the template. Each camera and lighting arrangement was mounted on the three docking piles to provide an optimum view of the docking cone guides of jacket during the final stages of positioning.

One of the main problems in the design and the installation of the jacket was the presence of a calcarenite formation in soil extending from a level about 46 m below mud line and prediction of its load bearing capacity. The twenty 102 inch piles were designed to be driven vertically through pile sleeves, into which the piles are subsequently grounted. For the foundation piles the MENCK MHU 1700 hydraulic hammer was utilized. This hammer was equipped with sensors for the measurement of ram stroke, hydraulic oil pressure and impact velocity. This data was reviewed and used to obtain hammer performance characteristics for subsequent back analysis of the blow count records so that design load bearing capacities could be confirmed. Eight conductor 30" pipe were installed by driving to a penetration of 65 meters with a diesel hammer.
For the piles figure 3 shows that the average blowcount never exceeded 50 blows per meter. This observed driving behaviour demonstrates that the frictional

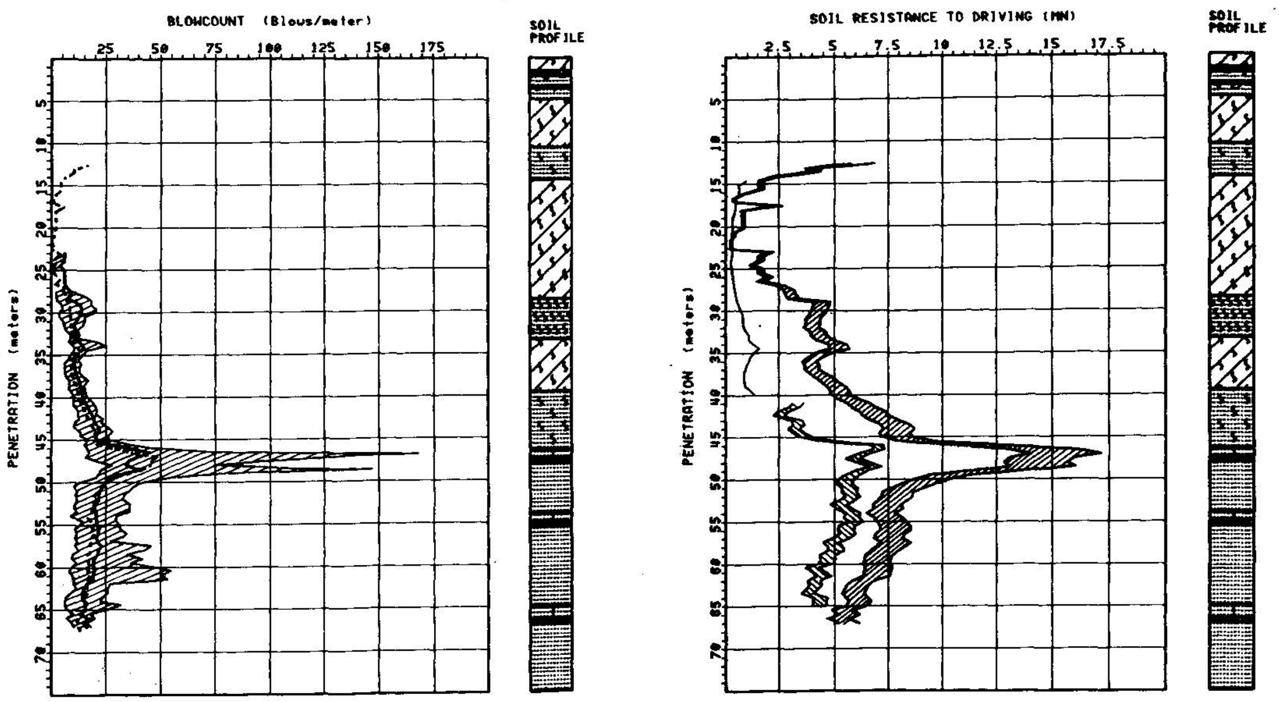


Figure 3 : Blawcount of foundation pile and soil resistance for pile and conductor during driving.

resistence acting on the piles was lower than expected due to remoulding and degradation effects, this also can be explained in terms of the excess energy available during driving.

The back analysis of soil resistence to driving is indicated in figure 3 with apparent difference in the driving behaviour of the piles and the conductors. For the piles, the increased resistance between 45 and 50 meters penetration appears to be completely lost shortly after penetration past the layer. The pile driveability behaviour was not fully understood. One feasible explanation that the length of strain wave generated by the MHU 1700 hammer reduces blowcount under easy driving conditions by maintaining relative movement between the pile and soil for a larger period.

The adequacy of the foundation however was confirmed by redrive tests, performed at a depth of 65 meters, the first after a set-up period of 148 hours, the second after a set-up period of 14 hours. These tests indicated that the set-up or strength gain of the Vega A soil was considerable, as a plotted on figure 4. From a soil resistence to driving of about 13 MN the driving resistence increased to 30 MN after 14 hours set-up, and about 45 MN after six days.

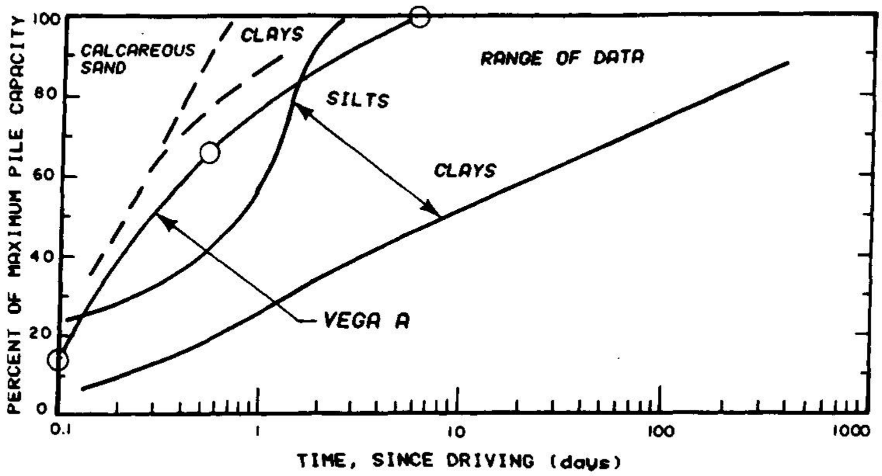


Figure 4 : Pile capacity for different soil conditions as a founction of time after driving.

Two piles were also monitored during driving by means of three transducers and three accelerometers mounted inside the pile at approximately 10 m from the top. Unfortunately the blows were recorded only to a penetration of 40 m and 46 m, after which values were not reliable since damage to the cables caused loss of insulation and excessive signal noise on some or all the instruments. However the calculation of pile bearing capacity and soil parameters by a stress wave program at those penetrations confirmed the indicated data of soil resistance.

3.3.2 Single Point Mooring

The SPM base/column assembly was towed out of Augusta horizontally and, 6 miles away in about 100 m water depth, uprighting of the column was started. After 6 hours the column was vertical and ready to be towed to the site. The most original feature of the installation work was the placing of 10,000 tons of iron ore inside the column and base drums. The iron ore was mixed with water to a form of a slurry and then injected through 4" hoses by means of centrifugal pumps. Once the column had settled on the sea bed, and the mooring had been secured by a large amount of excess water in the column, base ballasting was started. The ballasting procedure had been studied in order to be carried out without the aid of divers, and special underwater connections had been designed to be operated by ROV. To set the base skirts into harder soil layers, 6500 ton of iron ore were injected to reach a total foundation penetration of about 5.0 meters.

3.3.3 Sealine

The control of any pipelay vessel's position is a major factor in the installation of a subsea line. The pipeline route between platform and SPM are specified within well defined limits and positional tolerances, to permit the correct installation of the expansion spools. For this positioning a primary Microfix and secondary transponder systems were utilized. Microfix is a short range position fixing system combining the proven microwave interrogation techniques and results in a repeatable accuracy of 1 meter. Another essential aspect is the monitoring of the pull on the line and its geometry during laying operations. The touch-down point resulting from theoretical calculation of free span and barge heading position was continually monitored on the bridge.

4. IN-SERVICE MONITORING AND INSPECTION

A complete program for in-service inspection contained within a procedural manual for all activities of inspection and monitoring has been prepared and already issued. On this basis the structures are monitored by area and all data are recorded.

Structural and cathodic protection monitoring systems were designed and installed on the platform which continuously collect data to be used in the in-service annual inspection program defined for detection and control of cracks and/or malfunctioning.

4.1 Structural monitoring

The system installed on the platform comprises performance and environmental instrumentation.

The performance instrumentation (strain gauges and accelerometers) provides an evaluation of the performance of the structure relevant to the dynamic behaviour and fatigue during its operative life. The accelerations are measured at the top of the jacket, level + 18,70 m. The model frequencies of the platform can be found from the position of the peaks in the acceleration spectrum.

The axial stress in two selected members is measured by eight couple of strain gauges placed in the middle span and at the opposite side of the section. The strain gauges are connected to the data acquisition system panel and the axial strain is obtained by combining strain measured by each couple. The purpose of this stress instrumentation is to provide data which may be used

monitor fatigue. Strain gauges were selected to satisfy the following requirements: range $\pm$ 500 μm/m; accuracy $\pm$ 5 μm/m, measurement frequency range 0 to 0.8 Hz.
Environmental conditions are measured by two anemometers installed at the top of the drilling derrick and the living quarters and an acoustic type wave height measurement instrument.
For the SPM, the typical loads and motions can be considered as the addition of a component with low frequency and a component with high frequency varying with wave periods. The low frequency component is governed by a second order phenomena and is proportional to the square of the wave height, and its frequency is the natural frequency of the system (100 ÷ 400 sec.). The high frequency component is produced by the direct action of the waves on the structure, is proportional to the wave height, and its frequency is the wave frequency (3 ÷ 25 sec.). To monitor the SPM structures it is necessary to make the following measurements (see fig. 5) :
- the three angles between base/column, column/yoke and yoke/tanker by pitch-roll sensor;
- the load between the columns and tanker through the yoke by stress sensor.

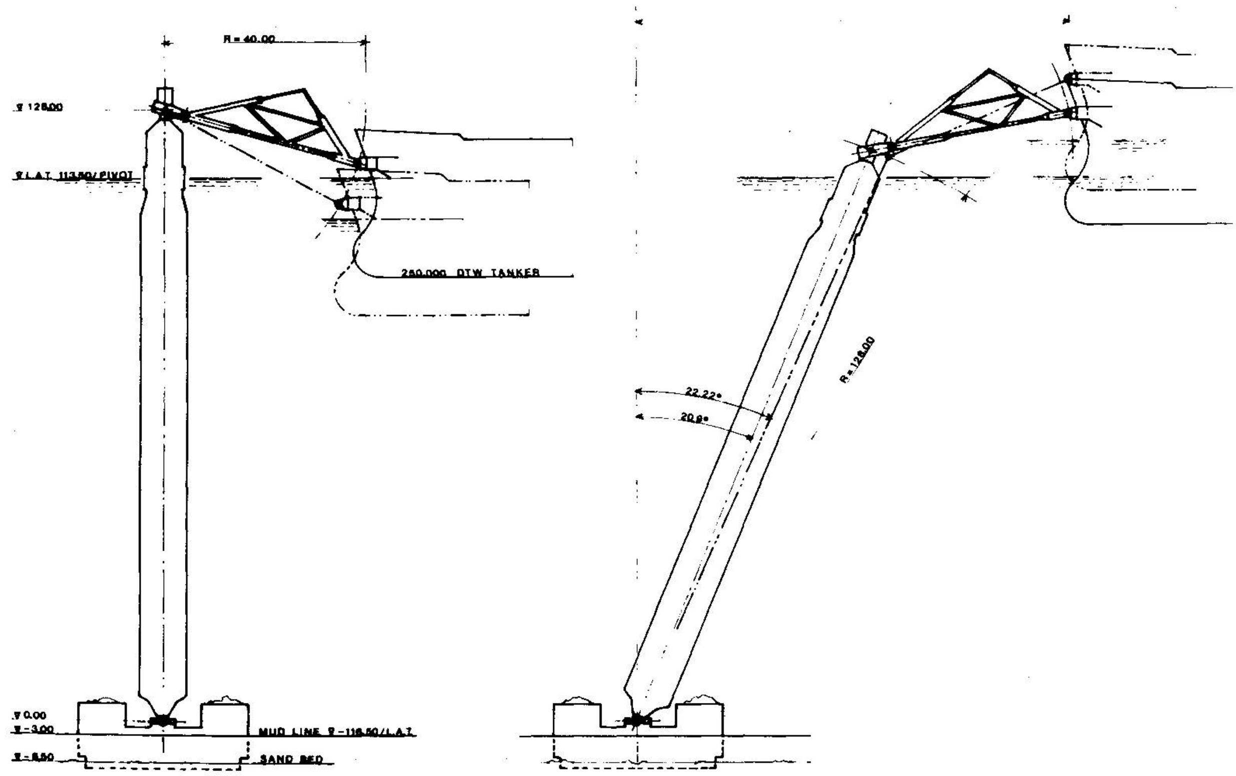


Figure 5 : Single point mooring movements of column, yoke and tanker

The implementation of this system has not yet been made and is currently under evaluation.

Another important problem in the in-service monitoring of SPM was the performance of the lower point bushings. These bushings are made of self-lubricating bronze material and have been designed for the life time of the system. To increase the confidence of the bushings an additional lubrication aid was provided to protect the mechanical parts and to monitor the wear of the self-lubricating material.

4.2 Cathodic Protection Monitoring

The permanent cathodic protection monitoring system is a means of supplying the state of metallic structures immersed in sea water during its operative life. Measurement of protection potential reached by joints, beams or other critical frames of the jacket allow the following:

- structure polarization course (first phase during calcareous deposit grouth) to be followed,
- control of the cathodic protection level reached and find out particular elements or zones which are under protected,
- perform in a timely manner the incidental retrofitting operations needed,
- collect data during the remaining period of operative life.

The influence of cathodic protection on the fatigue behaviour of a welded structure depends in a complex fashion upon the interaction of the mechanical, chemical and electrochemical parameters, on the crack initiation and the crack propagation [4]. Therefore the best way of improving fatigue life is still to delay crack initiation for as long as possible. Smooth shaped welds, post-weld improvement, and maintenance of a moderate potential are the best guarantee against the fatigue problem. During crack growth, moderate potential again provides the best compromise in reaching undesirable acceleration of growth rate.

The cathodic protection monitoring system is composed of:

- 30 underwater zinc reference electrodes, see figure 6,
- a control panel with a mimic diagram and the depolarizing unit.

4.3 Inspection Program

An inspection programme was designed to obtain information needed with regard to preventive maintenance of the structure and to assess the safety [5].

The inspection programme comprises:

- general visual inspection of the whole platform carried out every year at the beginning of the inspection period to discover any indication of deterioration which may require further inspection in that period,
- wall thickness measurement of jacket legs at water line,
- weld examination of a sample of selected highly stressed nodes to provide an early indication of distress.

Highly-stressed nodes were selected for periodic weld inspection according to the following criteria:

- computed fatigue life, less than 60 years, inspected by magnetic particle inspection once in each cycle of five years, with additional sampling by close visual inspection,

- computed fatigue life, less than 200 years, in conjunction with stress interaction ratio greater than 0.8, inspected once by magnetic particle inspection and once by close visual inspection in each cycle of five years,
- punching shear factor greater than 0.9, one node weld per jacket level was selected for close visual inspection once in each cycle of five years,
- stress interaction rate greater than 0.9 selected for close visual inspection once in each cycle of five years.

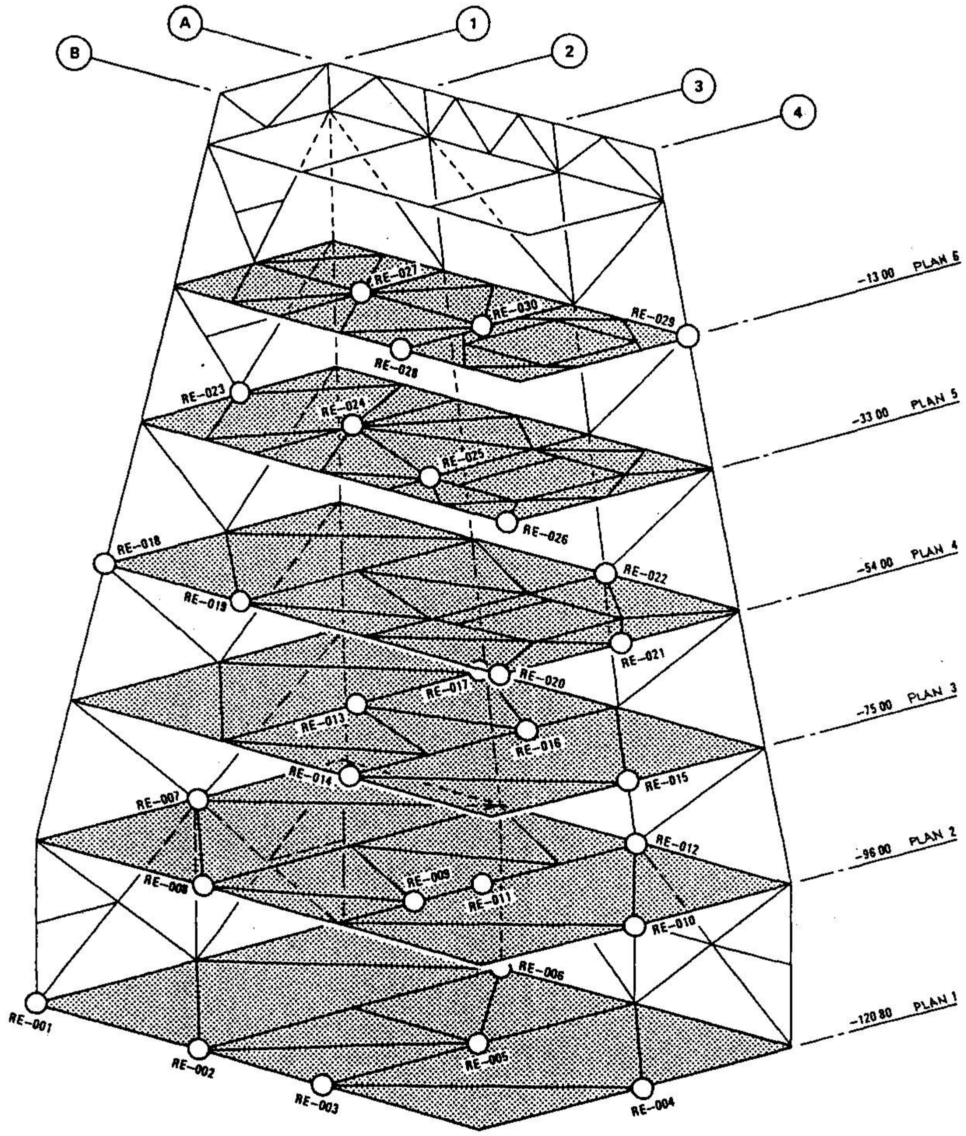


Figure 6 : Position of reference cells for cathodic protection monitoring system.

The inspection programme documentation lists inspection requirements during ordinary annual surveys, special surveys every five years and occasional survey after collision, eartquake or extreme storm.

5. CONCLUSIONS

During all phases of the Vega A project the main objective of inspection and monitoring activities was, and still is, to ensure the safety of the structure. Not only was personnel safety ensured, but also material losses or related problems were minimized.

Data obtained during construction and installation phases have been very interesting and will be useful for future similar projects.

The continuous monitoring and periodic in-service inspection programme assures continued safety and reduced maintenance costs.

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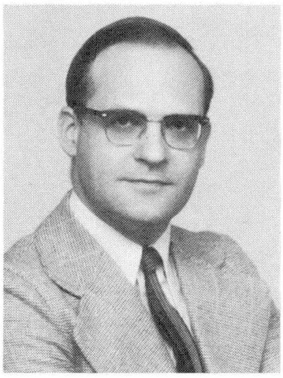
Long Term Monitoring of Bridges

Surveillance des ponts à long terme

Langzeitüberwachung von Brücken

Renaud FAVRE
Professor
Swiss Federal Institute of
Technology in Lausanne
Switzerland



R. Favre, born 1934, received his diploma in Zurich in 1957. He has worked in design offices and with a contractor in the field of concrete dams, bridges, and buildings. Since 1973, he has been Professor of concrete structures at the Swiss Federal Institute of Technology in Lausanne. He is also consultant of Wolf and Partners.

SUMMARY
Resulting from a mandate of the Swiss Federal Department of Transport, a proposition on how to organize the monitoring of bridges is presented. In particular the evolution of cracking and long term deflections have to be observed during the whole life span and reported graphically.

RESUME
Suite à un mandat du département fédéral suisse des transports, une proposition concrète est présentée concernant la surveillance des ponts. En particulier, les évolutions de la fissuration et des flèches à long terme doivent être observées pendant toute la durée de vie et reportées graphiquement.

ZUSAMMENFASSUNG
Gemäss einem Auftrag des eidg. Verkehrsdepartements wurde ein konkreter Vorschlag für den Brückenunterhalt ausgearbeitet. Insbesondere wird die Aufnahme der Rissentwicklung und der Langzeitverformungen eingegangen, welche für die ganze Lebensdauer der Brücke in grafischer Form festgehalten werden.

1. INTRODUCTION

Although the maintenance of the highway bridges in Switzerland is carried out under the responsibility of each Canton where the bridge is situated, the Swiss federal department of transport has the task of coordinating the work. Therefore this department gave the mandate to our Institute to propose a methodology of long term monitoring of bridges in order to be able to make a sound diagnostic during all their life time. The report has just been published [4]. After defining a clear (french) terminology, it gives a survey on disorders and damages encountered in bridges. It describes the present methods of control and presentes in more detail the most suitable ones. Finally it gives a concrete propositon on how to organize the inspections and the maintenance.

2. TYPES OF INSPECTIONS

For the long term monitoring of a bridge, a correct medium should be drawn between the occasional visits and the placement of a permanent observation team. This surveillance must be exercised by means of meticulous control actions during specific inspections. It has thus been proposed to split bridge monitoring up into
- periodic inspections every 5 years
- routine inspections every 15 months
- special inspections according to needs.

So there are 3 routine inspections between two periodic inspections. With such a timetable, 20 % of all bridges are subjected every year to a periodic and 60 % to a routine inspection.

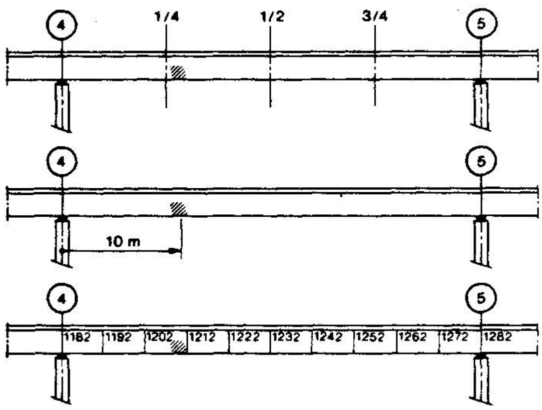


Fig. 1 Locating of observations

2.1 The routine inspections

The routine inspections should be done more or less every 15 months in order to get a change in season between two consecutive inspections, which may lead to the discovery of certain damages related to the climatic conditions.

The execution can be made by non specialised personnel of the highway department who have received an adequate training. The observations will be limited to a visual control of the bridge surface (cracks, humidity, corrosion) and of the equipment (bearings, joints, drainage).

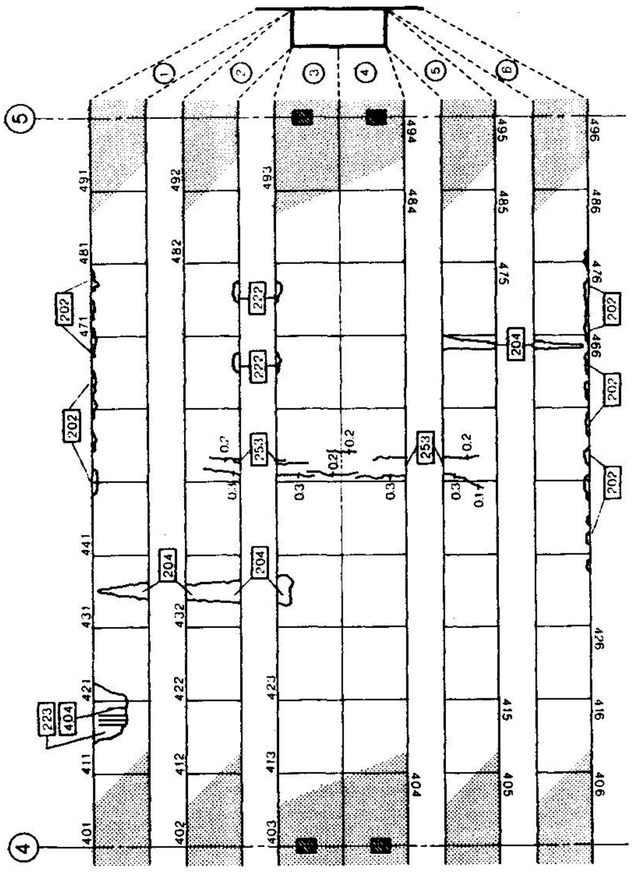


Fig. 2 Observations on the external surfaces plotted in a drawing provided with a numbered net

2.2 The periodic inspections

They have to give an image as complete as possible of the condition of the bridge and of its evolution. Cracks for instance are not necessarily a sign of disorder but their variation in time has to be known in order to be able to have a sound judgment. All areas of the structure must be controled. This means the necessity of putting the bridge out of service, either totally or partially, temporarily of for the whole duration of the inspection. The execution of the inspection requires specially trained personnel with sufficient access means (mobile foot bridges, nacelles etc.) : it should be possible to reach all points of the structure. Each periodic inspection should include :

- detailed visual control of the surface
- statement of the cracking patterns and their opening widths
- detailed control of the equipments

- measurement of :
  deck deflection
  displacements of the supports
  movements of foundations
- measurement of :
  E-modulus (sclerometer)
  depth of carbonatation
  presence of chlorides
  corrosion of reinforcement
- check for leaks of the watertightness.

A good deal of these observations should be noted in drawings (fig. 1). Cracks, water infiltration, traces of corrosion, bursting of concrete etc. have to be carried back conscientiously in the drawings in order to get a general survey of the disorders. For that purpose, the drawing of each span should be divided in equal parts, creating a regular wide-mesh net (maillage) which has also to be reported on the bridge itself. In fact, it proves difficult to situate an observation only by means of a distance, for instance 10 m from pier 4 as shown in Fig. 1. These drawings have to be utilized during all the life time of the bridge, so that with time a lot of indications will have to be plotted, with many specifications as crack length, crack width, date of appearance, evolution observed from the last inspection etc.

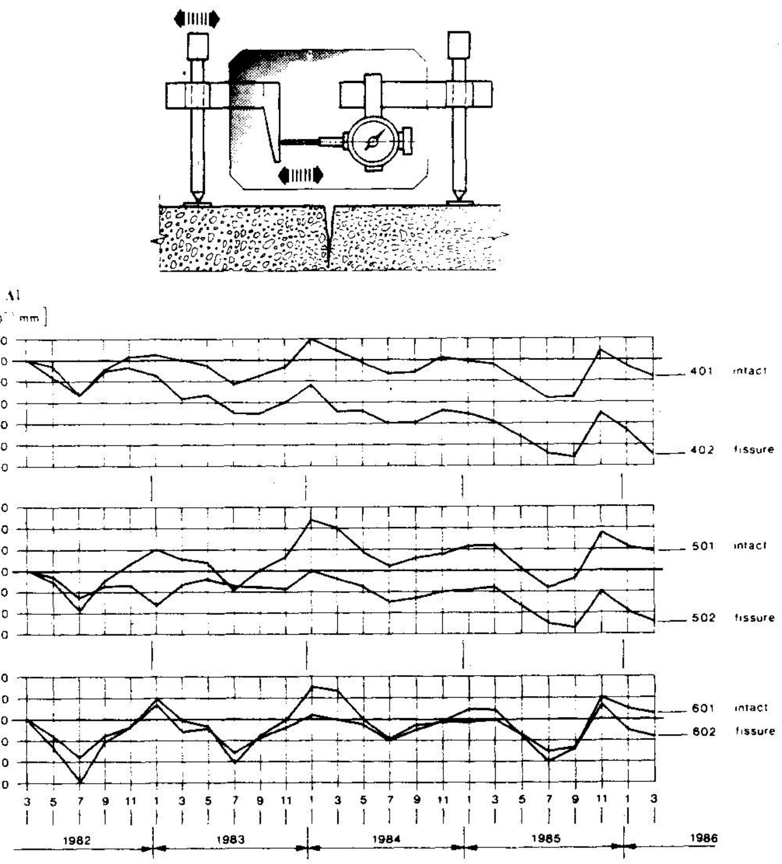


Fig. 3 Evolution of crack-openings

As a result one obtains drawings as shown in Fig. 2 where all visible parts of the bridge have to be treated.

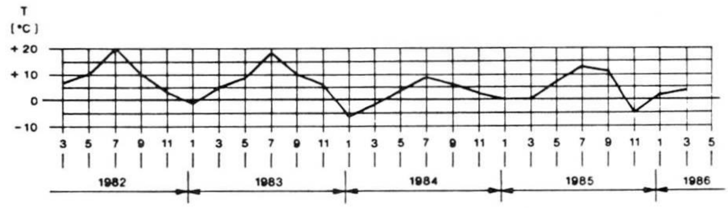


Fig. 4 Evolution of temperature in concrete

It is evident that it will be necessary to observe certain disorders in shorter intervals, for instance every two months (Fig. 3). From this figure, it can be seen that, in spite of temperature effects, crack Nº 602 seems to be stabilized whereas crack Nº 402 continues to open. In Fig. 4, the corresponding temperature in the neighbouring concrete is shown.

3. LONG TERM DEFORMATIONS

A most important and instructive assessment of the behaviour of a bridge will be obtained by monitoring the long term deformations. The most usual way will consist of optical or hydraulic levelling. The old known hydraulic levelling method by use of communicating vessels often fits especially well with the necessity of a regularly repeated observation during the whole life span of a structure.

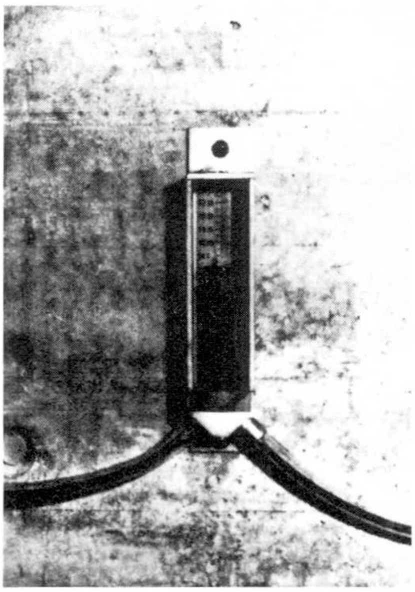


Fig. 5 Vessel for hydraulic levelling

Our Institute has been charged to install such communicating vessels in 10 bridges (Fig. 5 and 6). Having once installed these vessels, the cost for the observation of the displacements are very low so that it is recommended to install them in all important bridges, as far as the accessibility is provided.
A reference point has to be choosen, an abutment for instance. As a minimum, communicating vessels are placed on the supports and at mid-span, but also quarterly along the significant spans. In order to notably reduce the thermal effects, the measurements should be carried out in similar conditions, that is late autumn or winter. They therefore rarely coincide with the periodic inspections, at least at the beginning. They should be carried out every year for the first five, then every other year for the following four years and then every five years during the remaining life span. Thus the evolution tendancy can be better estimated.

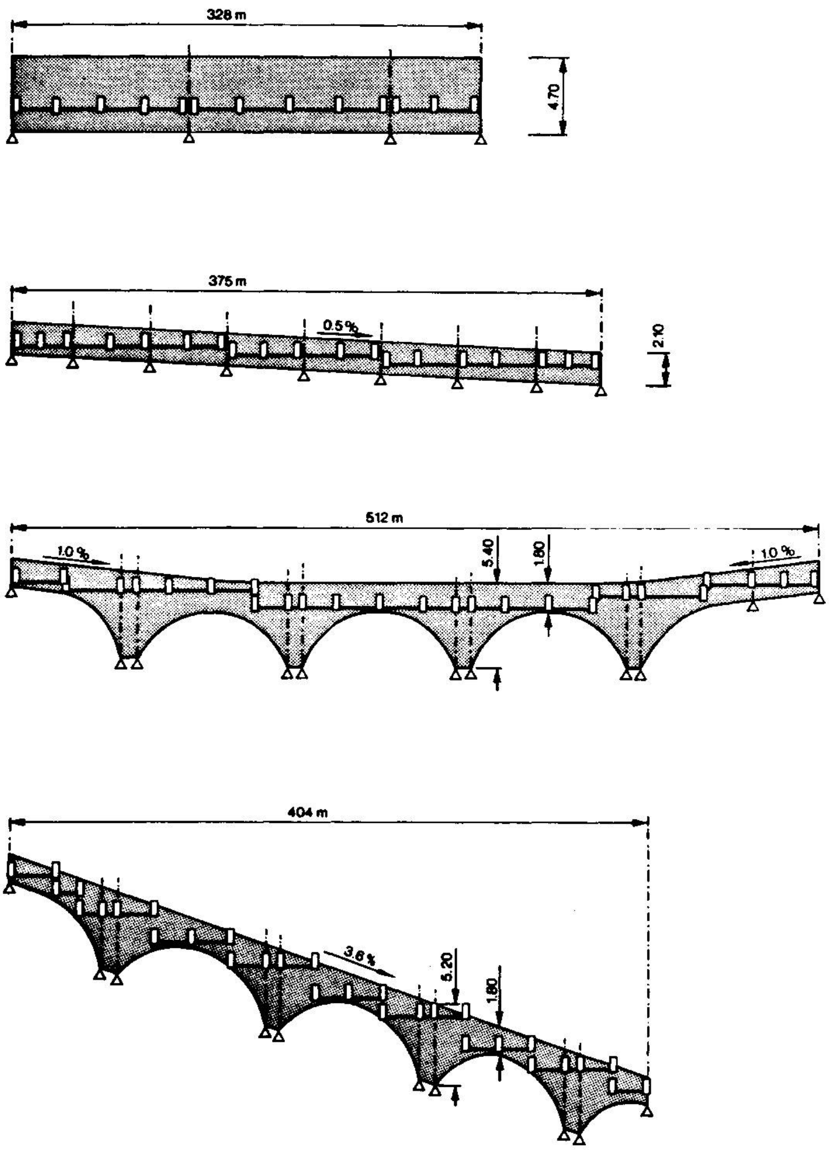


Fig. 6 Examples of installed communicating vessels

The results are interpreted under the following two aspects :
- affinity or not with elastic deformations
- size of the deformations compared with the predicted ones.

If the long term deflection curve has no good affinity with the calculated elastic curve, it may be the sign of a local weakness (Fig. 7). In the weak zone the increase of curvature cannot only be the result of creep but must be due to other effects, such as cracking, loss of prestressing force, bad concrete, bad continuity due to joints during execution etc.

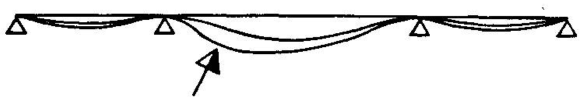


Fig. 7 Elastic and long term deflection curves without affinity

If the long term deformations are bigger than the predicted ones (Fig. 8), the reasons have to be investigated very thoroughly. One has to be aware of the fact that there is a real lack of knowledge concerning the prediction of long term deflections.

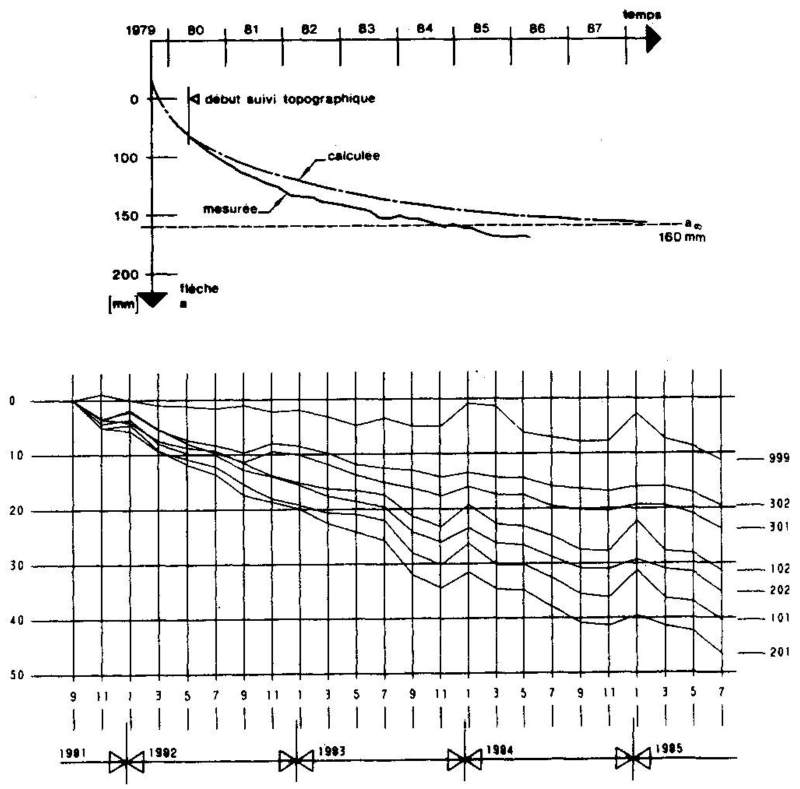


Fig. 8 Two examples of long term deflections greater than predicted

The real creep of the concrete in situ, the real prestressing forces, the moment redistribution due to asymetric reinforcement bars are difficult to be known with sufficient accuracy, so that even in the uncracked state I the prediction is not easy. But due to loads and especially thermal effects, some zones of the bridge may sometimes be cracked. That means that sometimes the curvatures and therefore the deflections will increase. When these effects have disappeared, the possibility of a certain irreversible amount of the deformations has to be considered. Tentative moment-curvature relationships for instantaneous reloading and unloading in the cases of simple bending and bending with normal forces are given in Fig. 9. These laws are given without any creep effect because they assume that the cracks are due to instantaneous effects.

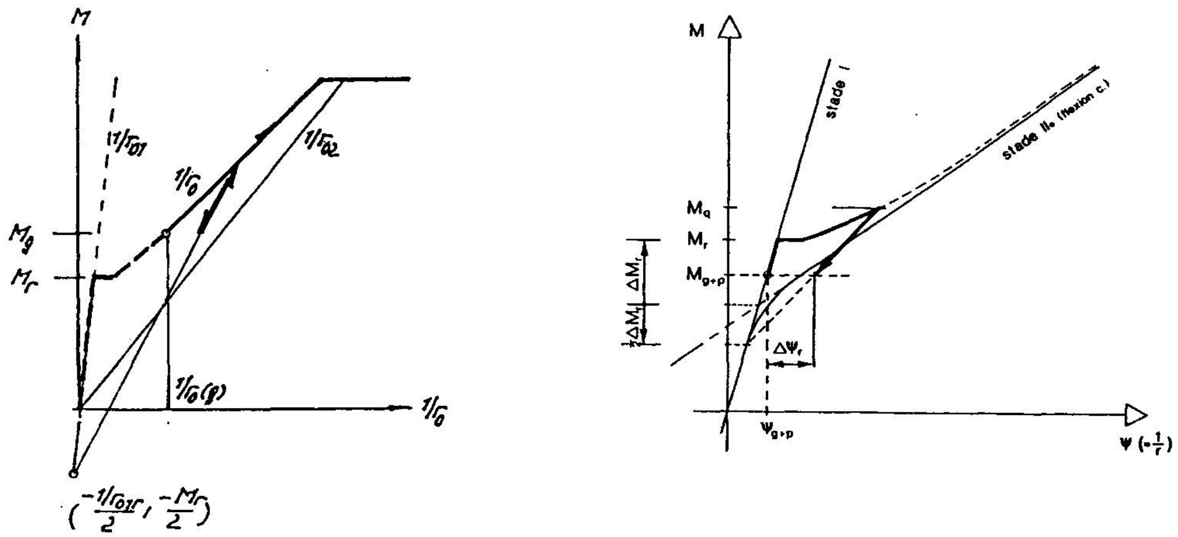


Fig. 9 Moment-curvature relationship for instantaneous reloading/unloading
a) simple bending
b) bending with normal force

The total deflections are given by the sum of long term deflections plus an irreversible part of often repeated instantaneous actions. It is absolutely necessary to increase our knowledge in that domain. In particular partially prestressed concrete bridges may suffer from these irreversible phenomena if they are frequently cracked due to thermal effects or large traffic loads.

CONCLUSION

This contribution shows a possible way of how to organize the long term monitoring of bridges through well defined systematic inspections. In order to have a complete view of the state of a bridge, it is necessary to have drawings in which the main observations are reported. Only with such a document an engineer will have a global information on the state of a bridge. The long term deflections are also of great importance but more information is needed on the real behaviour that should be expected. The big advantage of steel bridges are their elastic behaviour. Therefore it is absolutely indispensable to compensate the disadvantage of an unelastic long term behaviour in concrete bridges by better comprehension and forecast of their irreversible deformations.

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