Design and testing of corrosion damaged prestressed concrete joists: the Pescara Benchmark

A Di Evangelista¹, A De Leonardis¹, C Valente¹, L Zuccarino¹

¹Departement of Engineering, University “G. d’Annunzio” of Chieti-Pescara
Viale Pindaro, 42 – 65127 Pescara, Italy

E-mail: dievangelista@unich.it

Abstract. An experimental campaign named the Pescara benchmark and devoted to study the dynamic behaviour of corroded p.c. joists has been conducted. The steel corrosion reduces the area of the reinforcement and causes cracking of concrete so that r/c members are subjected to loss of strength and stiffness. It is of interest to evaluate the corrosion level at which the damage can be detected through signal processing procedures and how close such level is to the r/c member safety limits. Joists of current industrial production having different steel to concrete ratios are tested in different laboratory conditions. Dynamic tests involve either free vibrations and forced vibrations due to a moving mass simulating actual traffic loads in railway bridges. The paper discusses the rationale of the tests including the set up of the artificial corrosion, the static characterization of the joist and the dynamic tests in the different stages of corrosion experienced.

1. Introduction

Great attention has been paid in the literature to study the dynamics of mechanically cracked beam elements since several decades [1]. Theoretical solutions exist and specialized identification methods have been set up to estimate crack position and severity [2], [3], [4], [5]. Both homogeneous steel beams and composite reinforced concrete beams have been considered [6]. Comparatively less effort has been devoted to study the dynamics of prestressed concrete (pc) beams damaged by natural causes such as corrosion [7], [8], [9], [10]. This is a typical case for bridge structures where early damage detection in small vibrations range from traffic loads is the main concerns. These circumstances associated to the peculiar features of corrosion damage, as compared to mechanical cracking, make the identification far more challenging. In view of the above, the aim of the work is to make available, for subsequent data analysis and interpretation, a set of laboratory controlled experimental results representative of actual field conditions. To this end, a comprehensive experimental campaign has been carried out on artificially corroded pc joists. The paper reports the rational for the design and execution of the tests. A total of 15 joists (3 groups of 5 joists each) were corroded for 60% of the total beam length and dynamically tested. All the joists share the same geometry and concrete properties so that identical response is expected in the sound conditions. The three groups differ for the amount and position of the prestressing steel strands so that different damage entity and pattern are expected for the same corrosion level. All the joists were first tested in the sound state to define the reference
conditions, then they were subjected to accelerated corrosion and tested again at several damaged levels. The dependence of the results on summer-winter thermal conditions and the repeatability of the results among different tests has been considered and evaluated.

2. Tested joists
The joists used for the tests come from industrial mass production. They all have identical geometry, but differ for the amount and arrangement of the prestressing steel strands. These latter are composed by 2 or 3 wires by 2.25mm each. The joists length, \( L = 320 \text{ cm} \), has been set as the maximum compatible with laboratory room and equipment in order to identify the maximum possible number of natural frequencies in the linearity range of the adopted sensors. However, it should be highlighted that the joists actually differ from one another because of their rugged surface needed to improve the bond of the completion castings in the construction stages. Such differences vary along the joist length and are generally small, but not measurable and hence unknown. Therefore, in the following the nominal geometric properties will be used. The joists are grouped into three different sets: T1, T4 and T7, figure 1, where the homogenized area \( A_h \) and inertia moment \( I_h \) are reported together with the centroid of the section \( y_{gh} \) and the equivalent prestressing force eccentricity \( e \). These values are to be compared against the gross values \( A = 7200 \text{ mm}^2 \) and \( I = 4300000 \text{ mm}^4 \) referred to the concrete cross section that are conventionally assumed for computations. In order to quantify the expected changes between the two different assumptions the percent variations are given as: \( (A/A_h - 1)\times 100 = 1/4/7 \), \( (J/J_h - 1)\times 100 = 1.5/2/5 \) respectively for series T1, T4, T7. In engineering terms, the steel percentage of the three series is: 0.33%, 2x0.33%, 4x0.33% and the joists are then representative of small (T1) to high (T7) reinforcing steel percentage. Further, the number and arrangement of the steel strands at the lower face 2/3/5 for T1/T4/T7 favour different cracking mechanisms: spalling or localised cracking for T1 and delamination or diffused cracking for T7.

![Figure 1. Geometric and mechanical properties of the tested joists (dimensions in millimeters).](image)

2.1. Effect of the prestressing force
In the literature [11], [12], [13] studies can be found that deal with the problem of the dynamic behavior of a prestressed beam. These studies show the lack of significance of the prestressing force on the dynamic response of the beam. This problem is reconsidered here to appreciate the result that is important in the present context. A justification of the above can be obtained using the model of a small displacements Euler-Bernoulli beam under the hypothesis of an unbounded steel strand placed in the centroidal axis of the constant section beam. The dynamic behavior of a prestressed beam is governed by the couple of equations (1) where the index “c” stands once for “c” (concrete substructure) and once for “s” (steel strand substructure). The concrete is subjected to a constant compressive axial force \(-N\) whereas the steel strand is subjected to a tensile \(+N\). Further the compatibility conditions are guaranteed by the use of the same function \( w(x) \) of transversal displacements shared by both substructures (concrete beam and steel strand); whereas the global
equilibrium is guaranteed by the distributed load $\pm p(x,t)$ exchanged by both substructures with alternate sign.

$$E_i I_i \frac{\partial^4 w}{\partial x^4} \pm N \frac{\partial^2 w}{\partial x^2} + \rho A_i \frac{\partial^2 w}{\partial t^2} \mp p(x,t) = 0$$  \hspace{1cm} (1)

From equation (1) one gets:

$$EI + \rho \frac{\partial^4 w}{\partial x^4} + \rho A^+ \frac{\partial^2 w}{\partial t^2} = 0 \quad \text{and} \quad p(x,t) = N \frac{\partial^2 w}{\partial x^2} + EI \frac{\partial^4 w}{\partial x^4} + 2 \rho A^- \frac{\partial^2 w}{\partial t^2} = 0$$  \hspace{1cm} (2)

where: $EI = E_i I_i$, $\rho A^+ = \rho c A_c$, $\rho A^- = \rho s A_s$. The first equation in (2) shows that the beam vibrates according to a composite beam where both substructures contribute in parallel regardless the amount of the prestressing force $N$. The second equation in (2) quantifies the distributed load $p(x,t)$. As a consequence, the natural frequencies of a prestressed concrete beam are not directly influenced by the prestressing force $N$.

### 2.2. Effects of corrosion

Corrosion is an electro-chemical reaction that causes dissolution of steel into ferrous ions ($\text{Fe} \rightarrow \text{Fe}^{2+} + 2e^-$) with the creation of ferrous oxides (rust) and the loss of steel mass reduces the effective diameter of rebars and steel strands. The rust products have expansive character, 1 to 6 times the original mass, and exert an increasing pressure on the surrounding concrete. As the tensile strength of concrete is won cracking appears. The extent and morphology of cracking depend on a number of factors that comprise the amount/type of rust, the percentage of steel in the beam cross section $\omega$, the strands spacing $s$ and the thickness of the concrete cover $c$. In view of the above, the present case represents a challenging case since all the mentioned parameters, $\omega$, $s$ and $c$, contribute to resist to cracking. Comparatively large corrosion products are hence necessary to induce detectable damage in the joists. Here damage is either observable as surface cracking and invisible as microfracturing in the concrete volume. Further, as compared to mechanical cracking where cracks develop parallel to the cross section plane, corrosion cracking produces cracks in the direction of the rebars or steel strands. The transverse dynamics of the joists is slightly affected by this type of cracking and changes in vibratory properties are mainly related to microfracturing in the concrete volume rather than cracking. In these respects it is interesting to investigate to which extent conventional damage detection procedures can identify corrosion induced damage. A joint effect of corrosion is the reduction of bond between concrete and steel. When this happens the composite effect is lost and the rebars or steel strands slip and this become the dominant effect in the beam collapse. In order to prevent such mechanism the joists were corroded only in the central part in order to maintain the necessary anchorage length to the steel strands.

### 3. Accelerated corrosion

Natural corrosion is a very slow time process so that it is necessary to recur to accelerated corrosion. This method involves the use of an electrolitic solution and a dc power supply. The central part of the joist subjected to corrosion is immersed in a water solution at 5% of NaCl. The anode is constituted by the steel strands connected to the positive end of the power supply, whereas the cathode is made of copper wires diffused over and along the area to corrode. The arrangement is shown in figure 4 where it can be observed that the joists are turned over to ease percolation of the solution and favour the electrolitic process.

The Faraday’s law was used to have a theoretical relationship between the time over which the impressed current was allowed to flow and the extent of corrosion:
\[
\frac{I_{\text{corr}} \cdot t}{F} = \frac{\Delta w}{W_m/Z}
\] 

where \(I_{\text{corr}}\) is the electric current (Ampere), \(t\) the time (seconds), \(F\) the Faraday constant (96485 C/mol), \(\Delta w\) the consumed mass (g), \(W_m\) the molar mass of the iron (55.85 g/mol for Fe), \(Z\) the valency.

The Faraday’s law simulates the corrosion process through an ideal constant rate model of rust production. Even if the effectiveness of such model is known by the literature [14] the particular case of small diameter strands embedded into concrete and composed by steel wires in the presence of strong currents can alter the process either by modifying the valency exchanged or by localizing the chemical attack. This suggested to calibrate the procedure first on the strands alone and then on small pieces of joists. The calibration involved also the use of two different current intensity: low (10 \(\mu\)A/mm\(^2\)) and high (100 \(\mu\)A/mm\(^2\)). The agreement between the theoretical previsions and the actual results was satisfactory as shown in figure 2. A uniform corrosion was obtained either in the length of the strands and among the wires each strand is composed of, figure 4. The amount of corrosion is defined as the percentage \(C\%\) of the consumed mass \(\Delta w\) per unit length with respect to the original mass \(w_s\) of the sound strand. In the case of uniform corrosion this is equivalent to the area lost by each strand. This latter parameter is preferable to be referred to since it is directly related to the loss of strength of the joist, \(C\% = 100 \left(\frac{\Delta w}{w_s}\right)\). In table 1 the corrosion levels of each specimen are summarized. These were obtained using a constant current density of about 50 \(\mu\)A/mm\(^2\).

Table 1. Corrosion levels : \(C\%\) - Group T1,T4,T7

| Specimen | Group T1 | Group T4 | Group T7 |
|----------|----------|----------|----------|
|          | 1°step  | 2°step  | 1°step  | 2°step  | 1°step  | 2°step  |
| 1        | sound   | 6-15%   | sound   | 5%      | 15%     |
| 2        | 3%      | 3%      | 3%      | 15%     | -        | -        |
| 3        | 30%     | *       | 12%     | 12%     | 5%      | 15%     |
| 4        | 6%      | 15%     | 12%     | 12%     | 5%      | 10%     |
| 5        | 15%     | 15%     | 6%      | 15%     | -        | -        |
| 6        | -       | -       | -       | -       | -        | sound   |

*rupture of the specimen during testing

Figure 2. Apparatus for accelerated corrosion

Figure 3. Theor. (black) vs. exper.  
loss of mass in strands due to corrosion.
3.1. Corrosion cracking patterns
At the end of each corrosion phase and before the tests, a detailed map of the corrosion cracks in each specimen was drawn. The distribution and the widths of the cracks were measured. The most severe crack pattern, specimen T71, is reported in figure 5 according to both corrosion phases 5% and 15%. The analysis of the mapping shows that cracking is aligned with the direction of the steel strands. For low corrosion levels, $C\% < 5\%$, cracking can result either not observable or piecewise localised. In this latter case the crack width is not larger than $2/10$ mm. For higher corrosion levels, $C\% > 10-15\%$, the corrosion attack hampers quickly and the cracks coalesce and widen up to $15/10$ mm.

![Figure 5. Joist T71 (not in scale): crack maps $C\% = 5\%$ and $15\%$ and detailed picture at midspan.](image)

4. Experimental tests
Due to some unexpected results during testing of group T1 and T4, it was decided to perform a static characterization of the remaining group T7 before starting the dynamic tests. The static tests were conceived to give a preliminary and independent evaluation of the damage suffered by the joists.

4.1. Static tests
The same static scheme as per dynamic tests and a load pattern capable to provide a deformed configuration close to the first modal shape of the joists were adopted. The load was applied at midspan with steps $0.1$ kN according to three loading-unloading cycles of increasing intensity $0.4-0.8-1.2$ kN in a range of displacements inclusive of the dynamic deflected configurations. The static deformation was measured using 13 equally spaced displacement transducers. For all the load levels, the measured load-displacement curve shows essentially a linear behaviour. The comparison of the deflected configurations at loading and unloading shows small differences that vanish at increasing load. The joists state was assessed through the bending stiffness $\beta = EI$. The $\beta$ parameter was measured using three different methods: (i) slope of the linear regression on the load-displacement curve; (ii) average value of the secant stiffness at all load steps; (iii) curve fitting of the measured deflected configuration with the theoretical one. For reference, the estimated stiffness values for the sample joist T74 in the sound state are: (i) $\beta = 22.5 \cdot 10^7$; (ii) $\beta = 22.2 \cdot 10^7$ and (iii) $\beta = 21.8 \cdot 10^7$ kN/mm$^2$. It is also noted that the stiffness values estimated as (ii) have no uniform scatter, figure 6, the
dispersion goes inversely with the load level. The different stiffness at different load levels should be
taken into account when analysing the dynamic response if accurate estimates of the modal parameters
are required. The bending test was repeated in the corroded status. For safety, the maximum load level
was lowered to 0.8 kn with respect to the sound case. the load-deflection curves are compared in
figure 6. A slight stiffness reduction is observed. The average change of stiffness in the sample case
shown was. In view of the dynamic tests some comment can be helpful. First, given the slight changes
of $\beta$ between the sound and corroded state it will be to expect a far more reduced sensitivity in the
dynamic case when try to identify the frequencies. Second, no appreciable changes can be detected
between the sound and the corroded deflection shapes of the joists, the same is to be expected for
changes in the modal shapes.

4.1.1. Dynamic tests. Two types of dynamic tests were performed, namely: impulse (I) tests and
moving load (MV) tests that are summarized in table 2. In either case the joist under test has the static
scheme of a simply supported beam resting on steel supports with span length $L = 300$ cm. Three
different I tests were considered: (M) hammer, (R) sudden release of a point load, (C) sudden
settlement at one of the supports. The impulse was not measured. In the MV case two further spans
with the same static scheme were added before and after the instrumented joist to simulate the effects
of the incoming and exiting moving trolley as for a real bridge. The trolley speed was not measured.
Two different type of transit were considered: constant velocity driven by an engine and constant
acceleration obtained by the pulling force exerted by a calibrated falling body. Each test was repeated
a number of times by changing location and intensity according to table 2. Further the whole set of
tests was repeated twice for the same joist: one in the sound state and the other in the corroded state(s).
The dynamic response of the joists was recorded using 9 piezoelectric accelerometers and a sampling
frequency $f_{\text{sampl}} = 2000$ Hz. Two accelerometers were placed at both supports and the other 7 in the
positions given in figure 7. These positions realize a prefiltering of the dynamic response since they
 correspond to nodes or peaks of the first five modal shapes and therefore hamper or cancel the modal
contribution at a specific point.

Table 2. Dynamic tests.
### Impulse type tests

| Joist | Test type | Location | No. of intensity | No. of repetitions | Total no. tests |
|-------|-----------|----------|------------------|--------------------|-----------------|
| T1    | M: hammer | 0.2L, 0.3L, 0.4L | 2                | 2                  | 12              |
|       | R: release | 0.25L, 0.50L   | 2                | 2                  | 8               |
|       | C: settlement | 0.0L      | 3                | 3                  | 9               |
| T4    | M: hammer | 0.2L, 0.3L, 0.4L | 2                | 2                  | 12              |
|       | R: release | 0.25L, 0.50L   | 2                | 2                  | 8               |
|       | C: settlement | 0.0L      | 3                | 3                  | 9               |
| T7    | M: hammer | 0.2L, 0.4L     | 2                | 3                  | 12              |
|       | R: release | not performed |                  |                    |                 |
|       | C: settlement | 0.0L      | 2                | 3                  | 6               |

### Moving load type test

| Joists | Transit | Moving mass | No. of intensity | No. of repetitions | Total no. tests |
|--------|---------|-------------|------------------|--------------------|-----------------|
| T4     | acc = const | MV_S       | 1                | 3                  | 3               |
|        | vel = const | MV_D       | 2                | 3                  | 6               |

### Combined tests (impulse + moving load)

| Joists | Transit | Moving mass | No. of intensity | No. of repetitions | Total no. tests |
|--------|---------|-------------|------------------|--------------------|-----------------|
| T4     | acc = const | MV_S       | 1                | 2                  | 2               |
|        |          | MV_D       | 2                |                    |                 |

Figure 7. Sensors placement: (a) instrumented joist on supports; (b) sensors location (black dots), corroded joist length (black thick line) and modal shapes.

4.2. Moving load

Two different trolleys have been designed and realized to perform dynamic tests with moving mass (MV) figure 8. The shape of the cross section of the joist can be considered a sort of natural rail and was exploited in the trolleys design. However, because of the rugged surface of the joist, it was necessary to add some guides, external discs or internal tongues, to both trolleys in order stabilize the run. The trolley MV_S was designed to provide a point force in order to have data comparable with classical theoretical assumptions in some literature works on this subject. The trolley MV_D was designed with the purpose of getting changes of the moving mass. For this reason trolley MV_S is made by a single axle with two heavy metal wheels, whereas trolley MV_D is double axles C-shaped with hard rubber coated wheels. Further, the trolley MV_D houses a box in the top that can be filled...
with heavy material to vary the moving mass. The mass ratios between the trolleys and the joist are 1:5 for MV_S and 1:15, 1:10; 1:5 for MV_D. Another difference relies on the intrinsic stability of the two trolleys related to the axles position with respect to the trolley centre of mass. This can influence the noise of the run.

![Figure 8](image)

**Figure 8.** Moving loads: (a) trolley MV_S, (b) trolley MV_D, (c) trolley MV_D at launch

5. Results

The dynamic response of I tests was first processed using largely diffused conventional methods based on the Fourier spectral analysis. The aim was twofold: to check whether basic standard techniques are capable to deal with this kind and level of damage; to select the best set of records to be analysed in a second phase with more sophisticated identification techniques. A different research unit was charged to analyse the MV dynamic response. The first 5 frequencies and modal shapes were identified together with the equivalent viscous damping associated to the first mode (logarithmic decrement). It should be noted that the 5th frequency was at the bound of linearity range of the sensors. Two windowed sample records (sensors no. 3 and 5) of specimen T73 relative to C test no. 430 are reported in figure 9 together with the relevant FFT (all sensors). The time responses highlight how the higher order mode contributions damp down quickly. The FFT shows sharp peaks for the first two frequencies and peaks values progressively smoother and slightly dispersed for the higher order frequencies. Because of the particular disposition of the accelerometers only selected sensors were used to identify the different frequencies. The MAC values were used as possible indicators of changes of the modal shapes between different states: sound and damaged. The MAC values were very close to unity and were useless to detect damage in the present context. This result was expected in consideration of the symmetric and uniform damage distribution along the joists length. Identified frequencies and damping are given in figure 10. The graphs show the ratio of the identified values between the corroded and sound state. For frequencies, the reported values are the averages of all the tested joists at the maximum corrosion level. It is expected that the ratio of frequencies \( R_f \) is less than unity (loss of stiffness: \( \beta_{\text{dam}} < \beta_{\text{sound}} \)), the contrary for damping ratios \( R_d \) (increased energy dissipation). In the case of frequencies a systematic variation of \( R_f \) does not exist. \( R_f \) of group T4 comes out to be systematically greater than 1. This aspect should be investigate better. \( R_f \) of group T1 appears more stable, whereas \( R_f \) of group T7 suggests that the more sensitive frequencies are those with odd numbers. It is in fact conceivable some lack of sensitivity of the second frequency by comparing the modal shape with the length and position of the corroded part of the joists. However it should also been taken into account that the actual corrosion distribution can depart from ideal situation and localize. This happens for instance for joists of group 1 where \( R_f \) of \( f_2 \) is less than 1 and the lack of symmetry is reflected on \( f_3 \). In conclusion the results found can be physically interpreted if accompanied by the inspection of the damage suffered by the joists. But some randomness of the results do not help for blind identification. It should be studied if more sophisticated identification methods can overcome this limitation. The damping ratios \( R_d \) appears more stable and systematic (unless the case of joist T45). The values indicate an average increase of damping of about 20%, an entity safely usable for damage detection.
6. Conclusions
A large experimental campaign concerning the dynamic tests of mass production pc joist was carried out. Both free vibrations with different excitation and forced vibrations due to moving loads were considered. Group T1, T4 and T7 of 5 joists each were considered. The joist are geometrically identical and differs, group by group, by steel strands number and position. The tests were performed for both sound and corrosion damaged joists. The corrosion attack interested the central part of the joists; both ends of the joists remained sound to prevent bond slip. The corrosion process was artificial and accelerated according to the Faraday law. Essentially two corrosion levels were considered. The low level corresponds to localised capillary crack widths, whereas the high level corresponds to wide and distributed cracks. The results correspond to a first phase evaluation. In this case the free vibration records were processed using basic and conventional Fourier analyses. Inherent linear behaviour of the joists was assumed. This was confirmed by preliminary static tests. Modal parameters was then assumed as basic damage indicators. According to the above assumptions, it was found that low level corrosion does not produce detectable damage. Changes in modal parameters are observable for high level corrosion. The MAC index between the modal shapes in the sound and corroded state does not reveal effective because of the symmetry and uniformity of the damaged that do not alter significantly the vibration modes of the joists. As referred to the single test or joist, the frequencies show some dispersion. However, if the average values of all the available data of a given group are taken, some systematic trend is observed. Damping reveals the most sensitive parameter to corrosion damage. A detectable increase of energy dissipation is observed in the corroded state as compared to the sound state.
Acknowledgments
The work is part of the 2007 Italian national PRIN project BriViDi: Bridge Vibration and Diagnosis.

References
[1] Gudmunson P 1982 Eigenfrequency changes of structures due to cracks, notches or other geometrical changes Journal of the Mechanics and Physics of Solids 30(5) 39-353
[2] Rizos P F, Aspragathos N and Dimarogonas A D 1990 Identification of crack location and magnitude in a cantilever beam from the vibration modes Journal of Sound and Vibration 138 (3) 381-388
[3] Narkis Y 1994 Identification of crack location in vibrating simply supported beams Journal of Sound and Vibration 172(4) 549-558
[4] Das H Ch and Parthi D R 2010 Identification of crack location and intensity in a cracked beam by fuzzy reasoning International Journal of Intelligent Systems Technologies and Applications 9(1) 75-95
[5] Wang D S, Sheh D Y and Zhu H P 2009 Identification of Small Crack in Beam Structures Using Anti-Resonant Frequency and Wavelet Analysis Key Engineering Materials 413–414 63-70
[6] Valente C and Lamonaca B G 1997 Identification of cracked beams from the nonlinear dynamic response XIII AIMETA Congr., Siena 29 sept – 3 oct, Italy ed ETS Pisa, Italy 1997 Vol. IV 193-198
[7] Amborsini D, Luccioni B and Danesi R 2000 Theoretical-experimental damage determination in prestressed concrete beams The e-Journal of Nondestructive Testing & Ultrasonics 5 (7)
[8] Razak H A and Choi F C 2001 The effect of corrosion on the natural frequency and modal damping of reinforced concrete beams Engineering Structures 23 1126-33
[9] Torres-Acosta A A, Fabela-Gallegos M J, Muñoz-Noval A, Vázquez-Vega D, Hernandez-Jimenez J R and Martínez-Madrid M 2004 Influence of Corrosion on the Structural Stiffness of Reinforced Concrete Beams Corrosion 60 (9) 862-72
[10] Capozucca R 2008 Detection of damage due to corrosion in prestressed rc beams by static and dynamic tests Construction and Building Materials 22 (5) 738-46
[11] Kanaka K and Venkateswara G 1986 Free vibration behavior of prestressed beams, ASCE Journal of Structural Engineering 112 (7) 433–437.
[12] Saidi M, Douglas B and Feng S 1994 Prestress force effect on vibration frequency of concrete bridges, ASCE Journal of Structural Engineering 120 (7) 2233–2241.
[13] Hamed E and Frostig Y 2006 Natural frequencies of bonded and unbonded prestressed beams–prestress force effects, Journal of Sound and Vibration 295 28–39.
[14] Ballim Y and Reid J C 2003 Reinforcement corrosion and the deflection of rc beams – an experimental critique of current test methods Cement & Concrete Composites 25 625-632