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THE EFFECT OF PRESTRESS FORCE MAGNITUDE ON THE NATURAL BENDING FREQUENCIES OF PRESTRESSED CONCRETE STRUCTURES

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ABSTRACT

The effect of prestress force magnitude (N) and eccentricity (e) on natural frequencies (NFs, ω_n) of prestressed concrete (PSC) structures has been widely debated. No agreement currently exists as to how prestress force magnitude and eccentricity affect natural frequencies of PSC structures. Some reports state that the NFs decrease with increasing N ("compression softening" effect); others suggest that NFs increase with increasing N. Finally, some claim that the NFs are unaffected by N. This paper describes and analyses impact hammer testing conducted on a post-tensioned concrete beam at various levels of post-tensioning force. The aim of the research is to investigate how the NFs of PSC structures change with prestress force magnitude. Experimental modal analysis has been conducted and the experimental natural frequencies were identified by employing the Fast Fourier Transform (FFT). This was repeated for varying values of post-tensioning force at different impact locations. The applications of the results are widespread. The Eurocode 2 (EC2) equation for prestress loss as a function of time is well established. If the relationship between prestress force and NF for PSC structures can be established, then the variation of NF over the design life of a PSC structure due to prestress loss can be estimated. This problem is particularly pertinent in the field of post-tensioned concrete wind turbine towers and in the field of pre and post-tensioned concrete bridge girders.

KEYWORDS

Dynamics, fast fourier transform, impact hammer testing, natural frequency, post-tensioned concrete, prestress force, prestressed concrete, signal processing.

INTRODUCTION

The effect that the applied prestressing force has on the dynamic behaviour of PSC structures is a widely debated topic (Quilligan et al., 2012). There are currently three distinct arguments found in the literature;

1. The natural vibration frequency (ω_n) decreases with increasing prestress force magnitude (N). This is known as the "compression softening" effect and is based on classical Euler-Bernoulli theory of an externally axially loaded homogeneous beam (Miyamoto et al., 2000; Tse et al., 1978).
2. ω_n is unaffected by N. This is based on a non-linear kinematic model which concludes that the final equation of motion for a vibrating beam is independent of prestress force magnitude (Hamed & Frostig, 2006).
3. ω_n increases with increasing N. This has found to be the case in numerous empirical studies (Hop, 1991; Saiidi et al., 1994) however, a satisfactory mathematical model has yet to be formulated, despite some attempts (Kim et al., 2004).

The importance of this research is widespread. The effect of prestress force magnitude on NFs of PSC structures has many implications, specifically in the PSC bridge industry and for pre-cast, post-tensioned concrete wind turbine towers. Prestress force decreases over time due to concrete creep, steel relaxation, anchorage pull in and other factors. It should be possible to monitor or estimate changes in the NFs of PSC structures over the course of their design life to ensure their safety and serviceability. As a result, prediction of change in NFs of prestressed concrete structures over time is of great importance.

EXPERIMENTAL SET-UP

Impact Hammer Testing

Dynamic impact hammer testing was conducted on a post-tensioned concrete beam. The properties of the concrete beam tested are outlined in Table 1.

Table 1. Properties of concrete beam tested

Property	Value	Unit
Height, h	200	mm
Breadth, b	150	mm
Young's Modulus, E	26.88	GPa
Characteristic strength, f_{ck}	30	MPa
Density, ρ_c	23.55	kN/m^3
Span, l	2.0	m

Strength testing was also carried out on standard concrete cube specimens and an average of 42MPa was obtained, which is consistent with a characteristic strength of 30MPa (Table 1). Minimum reinforcement was provided in accordance with Eurocode 2. Cover to all reinforcement was specified as 25mm. 2No. H8 hanger bars were specified as top reinforcement, while 2No. H12 were specified as main (bottom) reinforcement. Shear links (H8) were provided at 200mm centres. Extra shear reinforcement was provided in the beam ends to prevent bursting failure due to localised stresses during post-tensioning, in accordance with the CIRIA method. 4No. H10 U-bars were provided at the beam ends for anchorage and continuity of reinforcement. A 20mm straight-profiled plastic duct was cast through the centroid of the concrete section, through which the post-tensioning strand was threaded. The estimated first natural frequency, taking into account the geometric effects of reinforcement with E_s = 205GPa, and post-tensioning strand on the second moment of area of the section was estimated to be 78.05Hz.

The experiment was set-up as shown in Figure 1. A 15.7mm Freyssinet prestressing strand, with a yield strength, f_y=1880MPa was threaded through the straight-profiled duct. A 20mm thick steel plate was placed against the rectangular beam face at each end. 300 ton hydraulic loading jacks were then placed at each end of the beam. The jacks were connected to load cells, which measured the magnitude of the post-tensioning force induced in the beam. The strand was secured using collets on either end of the load cells. An impact hammer rig was assembled using a rope and pulley system.

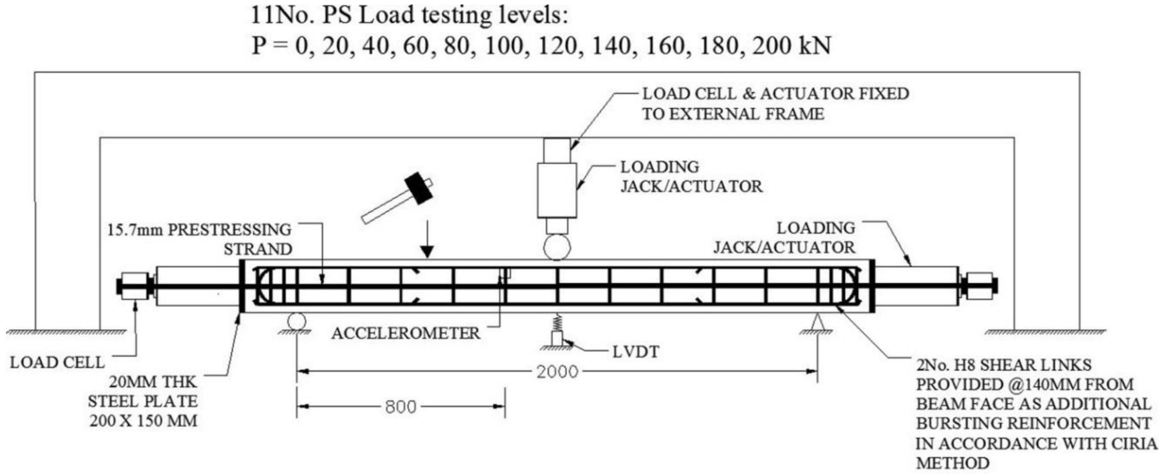


Figure 1. Experimental set-up 1

Impact hammer testing was conducted at three locations along the length of the beam span, as outlined in Figure 2, labelled L1-L3. The beam was struck 10 times at each location using an impact hammer rig assembled in the lab with a rope and pulley system. This helped ensure repeatability of the experiment. Strain gauges were affixed at the three impact locations (ε1- ε3) in order to obtain the mode shapes of vibration. The accelerometer (A1) was strategically placed at a distance of 800mm from the support, in order to identify all of the first three modes of vibration. Placement at midspan would eliminate the opportunity to obtain the second mode of vibration as it is a nodal point for the second mode. A fourth strain gauge (εa1) was placed in the axial direction, close to midspan, in order to compare the axial strain data with the prestress load data obtained from the load cells.

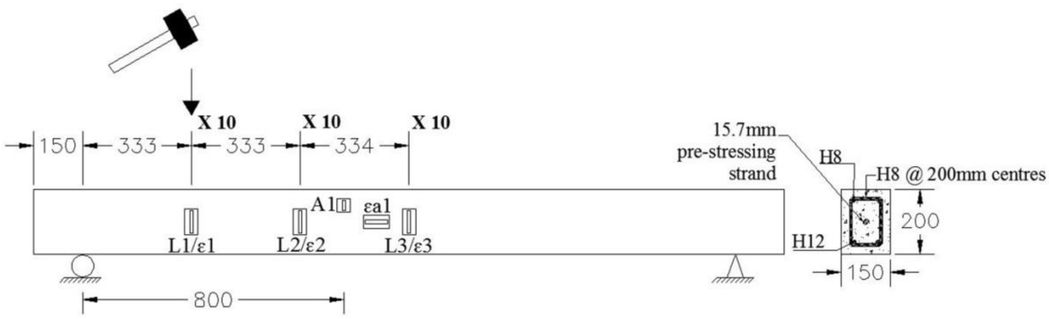
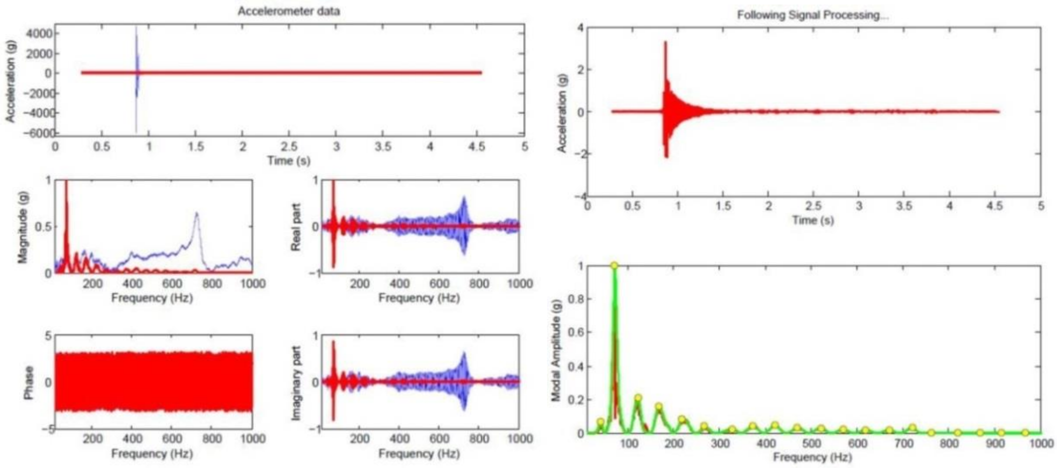


Figure 2. Instrumentation set-up on concrete beam

SIGNAL PROCESSING

The raw acceleration-time data obtained from a StrainSmart 6000 data acquisition system was analysed using a Fast Fourier Transform (FFT) in Matlab and the peaks of the absolute values of the transform were located in the frequency domain. It was found that the unprocessed signals were too noisy to correctly identify the correct modes of vibration. Noise was then removed from the signal through a signal processing technique developed by the authors. The signal was first *"centred"*, using a moving average filter. *"Zero drift"* was then removed from the signal, by shifting the signal so that it oscillated about zero. Electrical noise at 50Hz and all harmonics were removed using a "notch" filter. Then a 40Hz, 6th order high-pass butterworth filter was applied to the signals, to remove all low frequency components. The fundamental frequency was expected to be approximately 78Hz. Finally, the accelerometer data was then smoothed in the time domain to obtain the final processed input signal. Following this, the FFT was performed on the filtered signal to obtain the frequency response function (FRF). This FRF was then smoothed and the peaks were identified. Figure 3(a) shows the raw (unprocessed) signal in blue, in both the time and frequency domains, and also the filtered (processed) signal in red. As shown in Figure 3(a), the peaks are more readily identified using the filtered signal (red) rather than the raw data. Figure 3(b) shows the filtered signal in both the time and the frequency domain. The green line shows the signal after it has been smoothed in the frequency domain, and the peaks have been identified (yellow data points).

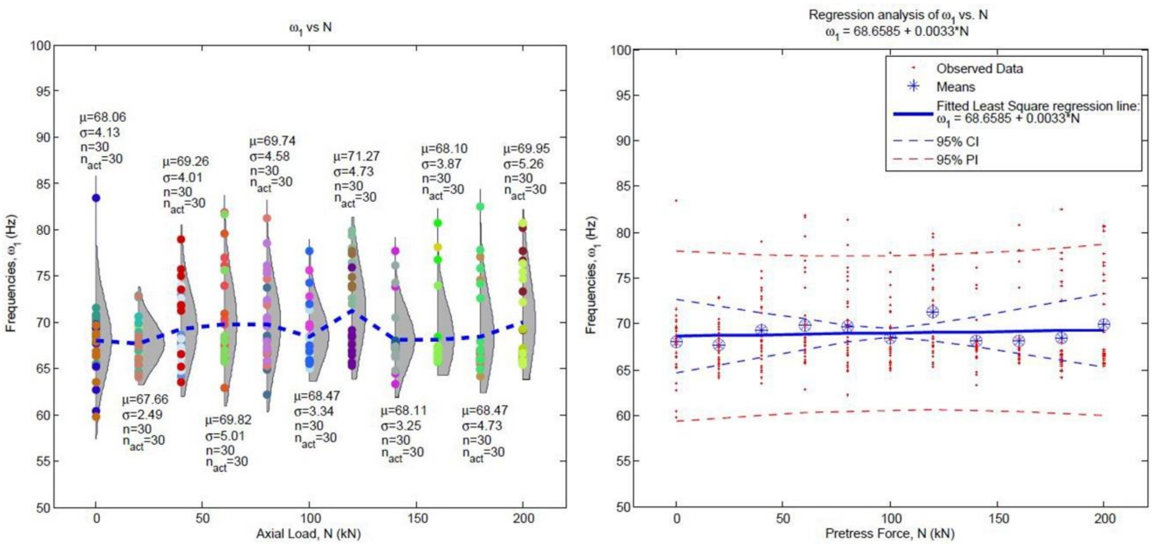


(a) Signal Processing (b) FFT and peak identification

Figure 3. Signal processing, FFT and peak identification

RESULTS AND DISCUSSIONS

Figure 4(a) shows the results of the experimental modal analysis of the accelerometer impact hammer data. 30 data points were collected at each prestress load level. Assuming normal distribution of the data points at each load level, the standard deviation of the data ranges between 2.49 and 5.26 Hz. The mean frequencies at each post-tension load level were joined (dotted blue line). No clear increasing or decreasing trend in natural frequency with varying post-tensioning force was observed. A linear regression analysis was carried out to determine if there is an increasing or decreasing trend in the frequency with changing post-tension force magnitude, Figure 4(b).



(a) Observed Data (b) Least square regression of data

Figure 4. Regression analysis of Frequency vs. Post tensioning force

The regression analysis indicates that there is no statistically significant change in fundamental frequency with increasing post-tensioning force (Table 2). The linear regression equation is given as;

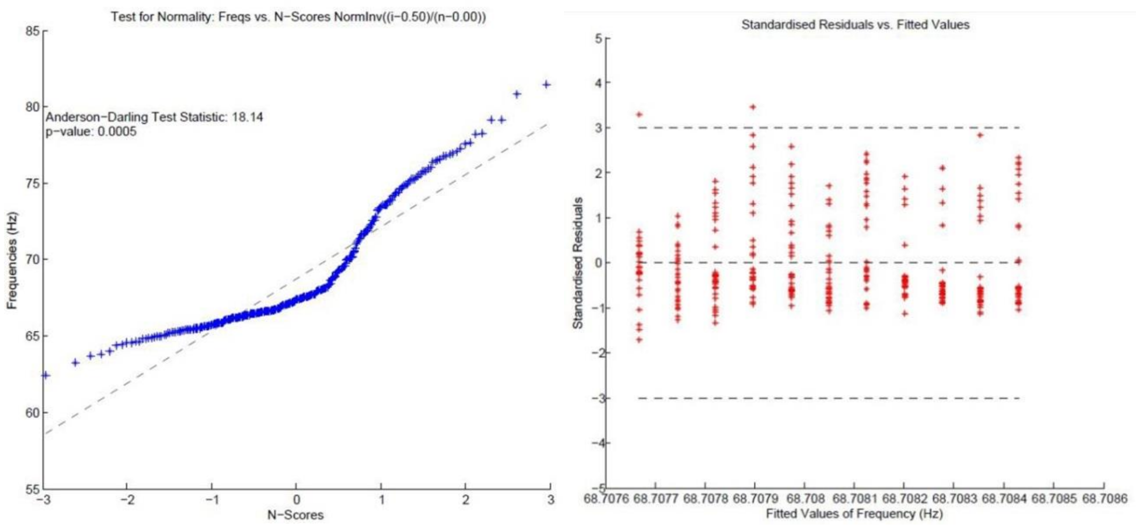
$$\omega_1 = 68.6585 + 0.0033N \tag{1}$$

ω_1 is the fundamental frequency in Hz (response variable) and N is the post-tensioning force in kN (predictor variable). The R^2 value is 2.44×10^{-3}, indicating no correlation between post-tensioning force and fundamental frequency. The R^2 parameter measures the goodness of fit of the data to the regression line. A R^2 value close to 1 represents a good fit, whereas a R^2 statistic close to 0 indicates little or no relationship between the regression variables. Table 2 shows 95% confidence intervals on the value of the linear regression parameters, intercept and slope. As indicated, we can strongly reject the null hypothesis that the long run value of the intercept is zero, and conclude that it lies between 67.2023Hz and 70.1147Hz. We do not reject the null hypothesis that the long run value of the slope parameter is equal to 0, and subsequently conclude that the fundamental frequency is independent of prestress force magnitude.

Table 2. Regression Parameters

Regression Parameter	**Value**	**Std. Error**	**t-value**	**t_c**	**p**	**95% CI**
Intercept	68.6585	0.6437	106.6604	2.2622	0.0000	(67.2023,70.1147)
Slope	0.0033	0.0054	0.6124	2.2622	0.2777	(-0.0090,0.0156)

Following a test on the normality of the data, as shown in Figure 5(a), it was shown that the data does not follow a normal distribution. The p-value is < 0.0005, and the assumption of normality of data must be rejected under a significance level of α=0.05. This brings the validity of the regression analysis into question, as one of the main prerequisites for conducting these analyses is the assumption of data normality. It is postulated that the lack of normality is due to the beam being tested over a two day period, and subsequently subject to temperature fluctuations that may affect the modal properties of the test specimen.



(a) Normality test (b) Standard residuals vs. fitted values

Figure 5. Residual analysis

Figure 5(b) shows the relationship between the standardised residuals and the fitted data. There is no reason, from this data to conclude that the long run standard deviation of the data is different for each fitted value. Furthermore, only two outliers have been identified (standardised residuals > +3 or < -3), at a significance level of α=0.05. For a data set containing 330 data points, this is not considered to be unusual.

CONCLUSIONS

From the experimental modal analysis conducted on the post-tensioned concrete beam with zero eccentricity, and the subsequent regression analysis, it can be concluded, from a statistical standpoint that, for a straight profiled prestress strand, no change in natural bending frequency was observed, and that the prediction of natural frequency is independent of prestress force magnitude over the given range of prestress force magnitudes. This concurs with the mathematical model outlined by Hamed and Frostig (2006). Saiidi et al. (1994) report that the natural frequency increased with increasing post-tensioning force, when they conducted similar studies, however, they did report the presence of cracking at midspan in their tested beam specimen, and speculated that the increase may be attributed to crack closure as the post-tensioning force increased. In this case, it was ensured that cracking did not occur. Furthermore, their study did not include a statistical perspective. It should be noted however, that statistical acceptance or rejection of the null hypotheses does not necessarily mean that comment can be made on what is occurring from a phenomenological perspective. More data is required in order to come to more definitive conclusions in this regard.

Future Work

This study will be expanded to include data for 9No. beams, each with different straight-profiled prestress strand eccentricities. The relationship between prestress force eccentricity and natural frequency will therefore be investigated. The existence of interaction effects between prestress force magnitude and eccentricity and their effect on natural bending frequencies will also be investigated.

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