PSC 거더교의 진동기반 긴장력 손실 모니터링

Vibration-Based Monitoring of Prestress-Loss in PSC Girder Bridges

김 정 태† 홍 동 수* 박 재 형* 조 현 만**

Kim, Jeong-Tae Hong, Dong-Soo Park, Jae-Hyung Cho, Hyun-Man

(논문접수일 : 2007년 10월 25일 ; 심사종료일 : 2007년 12월 12일)

요 지

본 논문에서는 프리스트레스 콘크리트(PSC) 거더교의 긴장력 손실을 예측하기 위한 진동기반 모니터링 체계를 제안하였다. 제안한 체계는 긴장력 손실 경보 단계와 긴장력 손실 정도를 평가하는 단계로 구성하였다. 먼저, 긴장력 손실 경보를 위해 두 위치에서 취득된 주파수 응답의 변화를 사용하여 긴장력 손실의 발생을 모니터링하는 새로운 전역적 손상경보기법을 제안하였다. 제안된 기법은 응답신호의 파워스펙트럼만을 이용하기 때문에 별도의 모드해석과정 없이 실시간으로 손상경보가 가능하다. 다음으로, 긴장력 손실 정도를 평가하기 위하여 고유진동수의 변화로부터 긴장력의 상대적인 손실 정도를 평가할 수 있는 긴장력 손실 예측 기법을 선정하였다. 제안된 체계의 유용성을 축소 모형 PSC 거더에 대한 실험을 통해 평가하였다.

핵심용어 : 진동기반 모니터링, 긴장력 손실, 손상검색, PSC 거더, 고유진동수, 가속도 응답

Abstract

A vibration-based monitoring system is newly proposed to predict the loss of prestress forces in prestressed concrete (PSC) girder bridges. Firstly, a global damage alarming algorithm is newly proposed to monitor the occurrence of prestress-loss by using the change in frequency responses. Secondly, a prestress-loss prediction algorithm is selected to estimate the extent of prestress-loss by using the change in natural frequencies. Finally, the feasibility of the proposed system is experimentally evaluated on a scaled PSC girder model for which acceleration responses were measured for several damage scenarios of prestress-loss.

Keywords : *vibration-based monitoring, prestress-loss, damage detection, psc girder, natural frequency, acceleration response*

1. Introduction

Recently, the interest on structural health monitoring(SHM) of prestressed concrete(PSC) girder bridges has been increased. For a PSC girder bridge, the prestress force in tendon is an important parameter that should be secured for its serviceability and safety against external loadings and environmental conditions(Lin, 1963; Nawy, 1996). The loss of the tendon's prestress force (hereafter, prestress-loss) occurs along the entire girder due to elastic shortening and bending of concrete, creep and shrinkage of concrete, steel relaxation and frictional loss, and damage or severing of prestress strands.

Unless the PSC girder bridges are instrumented at the time of construction, the occurrence of damage cannot be directly monitored and other alternative methods should be sought. Since as early as 1970s, many researchers have focused on the possibility of using vibration characteristics of a structure as an indication of its structural damage

† 책임저자, 정회원 · 부경대학교 해양공학과 교수
Tel: 051-620-6227 ; Fax: 051-628-8146
E-mail: idis@pknu.ac.kr
* 학생회원 · 부경대학교 해양공학과 박사과정
** 정회원 · 부경대학교 해양공학과 연구교수

• 이 논문에 대한 토론을 2008년 4월 30일까지 본 학회에 보내주시면 2008년 6월호에 그 결과를 게재하겠습니다.

(Adams *et al.*, 1978; Stubbs and Osegueda, 1990; Kim and Stubbs, 1995, Doebling *et al.*, 1998; Kim *et al.*; 2003a). Recently, research efforts have been made to monitor the change in modal properties in related to the change in prestress forces(Saiidi *et al.*, 1994; Saiidi *et al.*, 1996), to investigate the dynamic behaviors of prestressed composite girder bridges(Miyamoto *et al.*, 2000), and to identify the change in prestress forces by measuring dynamic responses of prestressed beams(Kim *et al.*, 2003b; Law and Lu, 2005).

In this paper, a vibration-based monitoring system is newly proposed to predict the loss of prestress forces in prestressed concrete(PSC) girder bridges. To achieve the objective, the following approaches are implemented. Firstly, a global damage alarming algorithm is newly proposed to monitor the occurrence of prestress-loss by using the change in frequency responses. Secondly, a prestress-loss prediction algorithm is selected to estimate the extent of prestress-loss by using the change in natural frequencies. Finally, the feasibility of the proposed method is experimentally evaluated on a scaled PSC girder model for which acceleration responses were measured for several damage scenarios of prestress-loss.

2. Prestress-Loss Monitoring Methods

Vibration-based health monitoring is the global SHM activity to acquire the information on the entire structure. This global SHM activity includes detecting damage and estimating structural safety by monitoring dynamic responses of the entire structure. Usually, vibration-based methods can estimate structural damage by utilizing the modal information obtained from limited sensor arrays; however, these methods cannot detect small incipient damage, but only detect particular forms of damage depending on sensor lay-out. Also, they need well-established baseline models to identify the perturbation from the monitored information.

Based on the above considerations, a prestess-loss monitoring scheme is designed as follows: 1) Alarming the occurrence of prestess-loss in PSC girder by using the change in frequency response functions, 2) Estimating the extent of the alarmed damage by using a frequency-based method, proposed by Kim *et al.*(2003b).

2.1 Global Damage Alarming Method using Frequency Response Functions

Damage occurrence may be detected using frequency response of the structure. The basic idea is that frequency responses are functions of the structural properties such as mass, damping and stiffness. Damage causes the change in structural properties, which, in turn, results in the change in frequency responses of the structure.

For the relationship between the input force and the output response of a structural system, the frequency response function, $H(f)$, is defined as follows(Bendat and Piersol, 1993):

$$H(f)=\frac{V(f)}{U(f)}=\frac{1}{-m(2\pi f)^2+ic(2\pi f)+k} \tag{1}$$

where $U(f)$ and $V(f)$ are, respectively, force and displacement transformed into frequency domain. Also, m, c and k are, respectively, mass, damping and stiffness of the structural system.

Assume an harmonic force, $U_j(\omega_k)$, is excited with the kth natural frequency ω_k and with same magnitude and is applied at m locations($j=1,\ldots,m$) at the same time. Also, on assuming that noises, $n_i(t)$ and $n_{i+1}(t)$, generated at locations i and $i+1$ are uncorrelated with each other and with the force $u(t)$ into time-domain, then the displacement ratio between two outputs at locations i and $i+1$ is given as follows:

$$\frac{V_i(\omega_k)}{V_{i+1}(\omega_k)}=\frac{\sum_{j=1}^{m}H_{i,j}(\omega_k)U_j(\omega_k)}{\sum_{j=1}^{m}H_{i+1,j}(\omega_k)U_j(\omega_k)}=\frac{H_i(\omega_k)}{H_{i+1}(\omega_k)} \tag{2}$$

Furthermore, a frequency-response-ratio(*FRR*) function between the locations *i* and *i+1* is defined as

$$FRR_{i,i+1}(\omega_k) = \frac{S_{i,i}(\omega_k)}{S_{i,i+1}(\omega_k)} = \frac{E[V_i^*(\omega_k)V_i(\omega_k)]}{E[V_i^*(\omega_k)V_{i+1}(\omega_k)]} = \frac{H_i(\omega_k)}{H_{i+1}(\omega_k)} \quad (3)$$

where $S_{i,i+1}(\omega_k)$, $S_{i,i}(\omega_k)$ are cross-spectral and auto-spectral density functions, respectively and superscript * is the conjugate. By comparing a frequency-response-ratio measured at an undamaged baseline state to the corresponding one at a subsequent damaged state, a frequency-response-ratio assurance criterion(*FRRAC*) can be defined as follows:

$$FRRAC(b,d) = \frac{\{FRR_b^T FRR_d\}^2}{\{FRR_b^T FRR_b\}\{FRR_d^T FRR_d\}} \quad (4)$$

where the subscripts *b* and *d* denote the undamaged baseline state and its corresponding damaged state, respectively. Equation (4) represents the linear relationship between the pre-damaged frequency-response-ratio, FRR_b, and the post-damage frequency-response ratio, FRR_d. The *FRRAC* remains close to 1.0 if no damage. Otherwise, the *FRRAC* decreases from 1.0 if damage occurred(i.e., changes in physical properties *m*, *c* and *k*).

2.2 Prestress-Loss Estimation Method using Modal Parameters(Kim *et al.*, 2003b)

Kim *et al.*(2003b) proposed an equivalent flexural rigidity model using a tension-strength model as schematized in Fig. 1. Consider a simply-supported, uniform cross-sectional, PSC beam with a straight concentric tendon. Suppose that the beam is in axial compression due to the prestress loads applied at the anchorage edges. Then we can model that the beam is initially deformed in compression up to the deformed span length L_r(i.e., reduced in span length by δL) and the tendon is still in tension due to the constraint(i.e., anchoring) after elastic stretching for prestressing effect. The governing differential equation for the beam is expressed by:

$$\frac{\partial^2}{\partial x^2}\left(E_r I_r \frac{\partial^2 y}{\partial x^2}\right) + m_r \frac{\partial^2 y}{\partial t^2} = 0 \quad (5)$$

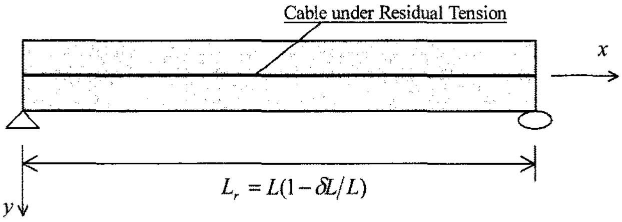
where $E_r I_r (= E_c I_c + E_s I_s)$ represents composite flexural rigidity of PSC beam section, in which $E_c I_c$ is flexural rigidity of concrete beam-section and $E_s I_s$ is equivalent flexural rigidity corresponding to the contribution of the tendon on the flexural resistance. Also, $m_r (= \rho_c A_c + \rho_s A_s)$ is the mass per unit length of PSC beam that combines the mass of concrete beam, $\rho_c A_c$, and the mass of the tendon, $\rho_s A_s$.

By equating modal characteristics of a cable under tension load N to those of a beam with equivalent flexural stiffness, the equivalent flexural rigidity of tendon is derived as follows(Kim *et al.*, 2003b):

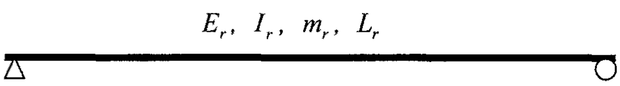
$$E_s I_s = \left(\frac{L_r}{n\pi}\right)^2 N \quad (6)$$

Applying Eq. (6) and appropriate boundary conditions to Eq. (5) leads the nth natural frequency, ω_n, of the residual-tension model as follows:

$$\omega_n^2 = \left(\frac{n\pi}{L_r}\right)^4 \frac{1}{m_r}\left(E_c I_c + \left(\frac{L_r}{n\pi}\right)^2 N\right) \quad (7)$$



(a) PSC beam under prestressed deformation



(b) Beam of equivalent flexural rigidity

Fig. 1 Equivalent flexural rigidity model of PSC beam (Kim *et al.*, 2003b)

As an inverse solution of Eq. (7), prestress forces can be identified as:

$$(N)_n = \omega_n^2 m_r \left(\frac{L_r}{n\pi}\right)^2 - E_c I_c \left(\frac{n\pi}{L_r}\right)^2 \tag{8}$$

By assuming no change occurs in beam's geometry and mass properties due to the change in prestress forces, the first variation of the prestress force can be derived as:

$$(\delta N)_n \cong \delta\omega_n^2 m_r \left(\frac{L_r}{n\pi}\right)^2 - \delta(E_c I_c)\left(\frac{n\pi}{L_r}\right)^2 \tag{9}$$

where, $(\delta N)_n$ is the change in prestress force that can be identified by the nth mode; $\delta\omega_n$ is the variation in nth natural frequency due to the prestress-loss; and $\delta(E_c I_c)$ is the change in flexural rigidity of concrete beam section. By relating Eq. (8) to Eq. (9), the relative change in prestress force, which identified from the nth mode, is obtained as(Kim *et al.*, 2003b):

$$\left(\frac{\delta N}{N}\right)_n = \frac{\delta\omega_n^2 - \dfrac{\delta(E_c I_c)}{m_r}\left(\dfrac{n\pi}{L_r}\right)^4}{\omega_n^2 - \dfrac{E_c I_c}{m_r}\left(\dfrac{n\pi}{L_r}\right)^4} = \frac{\delta\omega_n^2 - \delta\varpi_n^2}{\omega_n^2 - \varpi_n^2} \tag{10}$$

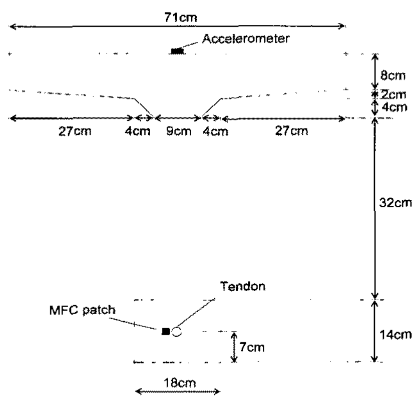
where ϖ_n is the nth natural frequency of the beam with zero prestress force and $\delta\varpi_n$ is the variation in ϖ_n due to $\delta(E_c I_c)$. From Eq. (10), the relative change in prestress force, $\delta N/N$, can be estimated by measuring the change in natural frequencies of the PSC beam. Eq. (10) can be simplified by further assuming no change in concrete flexural rigidity occurred due to the prestress-loss, i.e., $\delta\varpi_n \approx 0$.

3. Experimental Verification

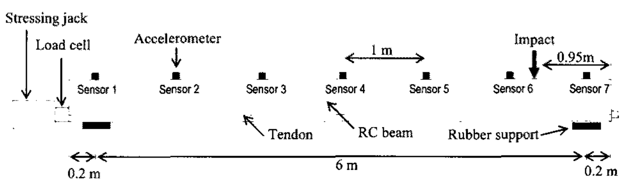
3.1 Test Structure and Experimental Setup

The schematic of the test structure is shown in Fig. 2. The tested girder spans 6.0m and installed on testing frame. Its both ends are simply supported by thin rubber pads. The T-beam section girder was reinforced longitudinally and in transverse direction with reinforcing bars with nominal 10mm diameter(equivalent to Grade 60). The stirrups were used to facilitate the positioning of the top bars. A seven-wire straight concentric monostrand with nominal 15.2mm-diameter(equivalent to Grade 250) was used as the prestressing tendon. The tendon was placed in a 25mm-diameter duct that remained ungrouted. The concrete was made of normal portland cement and had the maximum aggregate size of 25mm. The 28-day compressive strength of concrete was 23.6MPa and the mass density was about 2400kg/m^3.

As shown in Fig. 3, the structure was tested in a lab(Smart Structure engineering Lab) located at Pukyong National University, Busan, Korea. A series of tests were performed on the PSC girder from 23 to 25 January, 2007. The lab was air-conditioned to keep temperature and humidity close to constant during the tests. Room-air temperatures ranged from 20.4℃ to 21.5℃. Temperatures of the PSC girder had little variation as follows: the top surface 18.1~19.3℃, the middle web surface

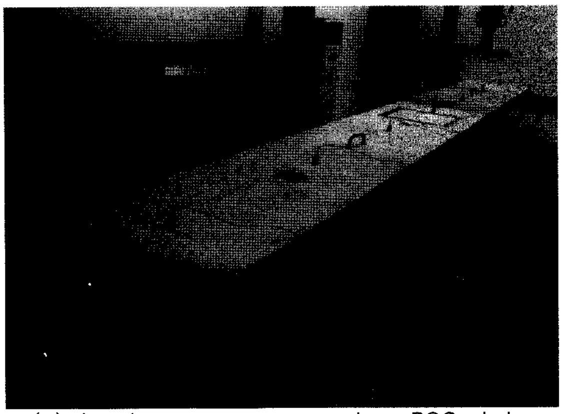


(a) Girder cross-section

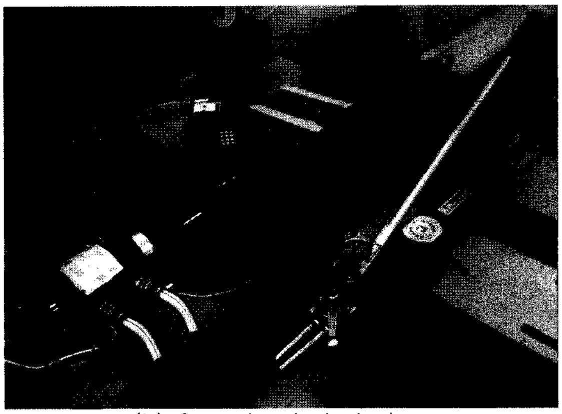


(b) Geometry of PSC girder and experimental setup

Fig. 2 Schematic of prestressed concrete girder

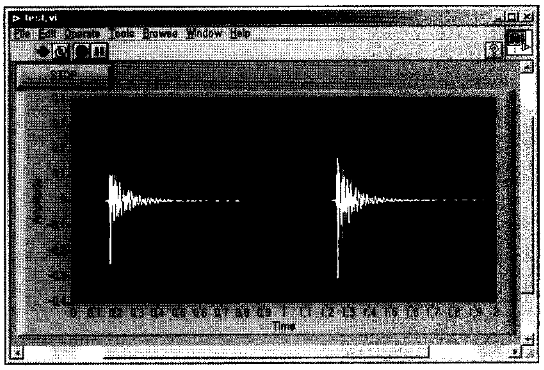


(a) Accelerometers mounted on PSC girder

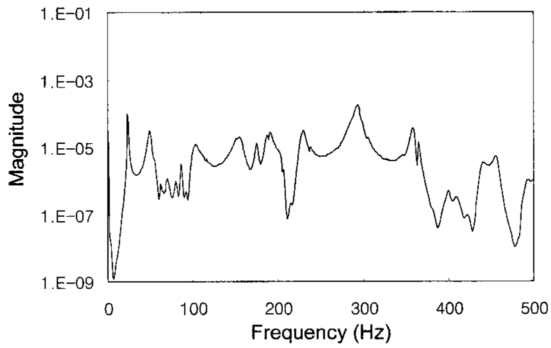


(b) Stressing jack devices

Fig. 3 Experimental setup in PSC girder

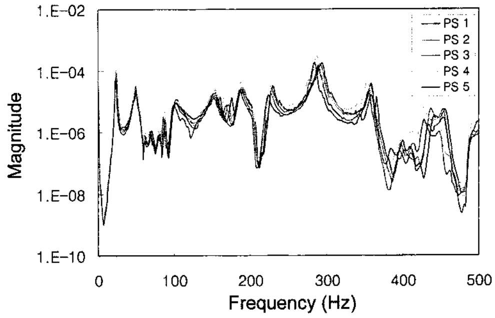


(a) Acceleration signal

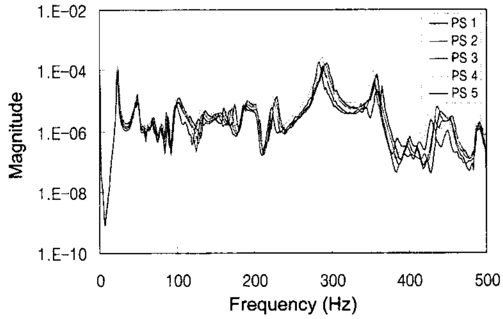


(b) Frequency response function

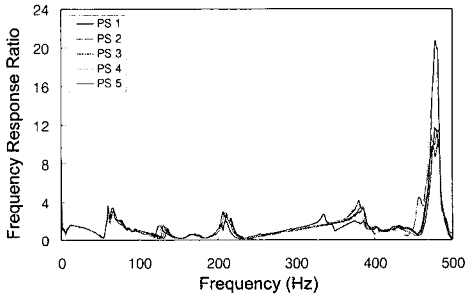
Fig. 4 Acceleration signal and frequency response function of PSC girder



(a) FRF(2): Sensor 2



(b) FRF(3): Sensor 3



(c) FRR(2,3) = FRF(2)/FRF (3)

Fig. 5 Frequency response functions for five prestress levels (PS1-PS5)

17.6~18.7℃, and the bottom surface 17.0~18.2℃, which were measured by using K-type thermocouple wires and PXI-4351 Temperature Logger.

Locations and arrangements of sensors on the test structure were designed as shown in Fig. 4. For acceleration measurement, seven sensors(Sensors 1-7) were placed along the girder with constant intervals and the impact excitation was applied at a location 0.95m distanced from the right edge. A type of ICP accelerometers were used in the test: PCB 393B04 with a nominal sensitivity of 1V/g and a specified frequency range(±5%) of 0.06~450Hz. The accelerometers were connected to the magnetic blocks which were attached to steel washers bonded on the top surface of the girder.

Impact forces, which were not controlled nor re-

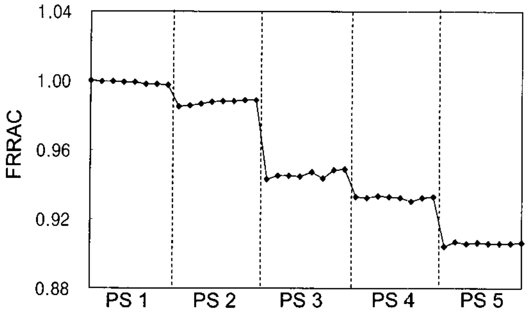


Fig. 6 Global damage alarming of prestress-loss occurrence

corded, were applied to the PSC girder by dropping a 3kg-weight hammer. Then dynamic responses in vertical direction were measured at the 7 Stations with a sampling frequency of 1 kHz. It was always positioned on Sensor 3 as the reference channel. The data acquisition system included a 16-channels PXI-4472 DAQ, a PXI-8186 Controller, and a PC with LabVIEW and MATLAB. It was set up to acquire signals from the accelerometers and furthermore to extract frequency response functions and modal parameters by frequency-domain decomposition(FDD) technique(Brinker *et al.*, 2001; Yi and Yun, 2004). As shown in Fig. 6, acceleration signals were measured from the PSC girder and frequency response functions were extracted by using the FDD method.

3.2 Damage Monitoring for Prestress-Loss

Axial prestress forces were introduced into the tendon by a stressing jack as the tendon was anchored at one end and pulled out at the other. A load cell was installed at the left end to measure the applied prestress force. Each test was conducted after the desired prestress force has been applied and the cable has been anchored. During the measurement, the stressing jack was removed from the structure to avoid the effect of the jack weight on dynamic characteristics of the test structure. The prestress forces were applied to the test structure up to five different prestress levels(i.e., PS 1 - PS 5) as summarized in Table 1. There were four prestress-loss cases between the five prestress levels.

Table 1 Natural frequencies measured for five prestress levels

Prestress Level	Prestress Force (kN)	Natural Frequency (Hz)			
		Mode 1	Mode 2	Mode 3	Mode 4
PS 1 (reference)	117.6	23.72	102.54	228.87	294.32
PS 2	98.0	23.60	101.70	227.16	291.92
PS 3	78.4	23.39	101.65	225.93	288.54
PS 4	58.8	23.23	101.39	224.20	287.59
PS 5	39.2	23.08	98.73	221.76	284.09

The maximum and minimum prestress levels were set to 117.6kN and 39.2kN, respectively. For each prestress-loss case, the prestress force was uniformly decreased by 19.6kN.

3.2.1 Global Damage Alarming by FRRAC

In Step 1, the occurrence of damage was alarmed in global manner by using the *FRRAC* described in Eq. (4). For each of the five prestress levels, acceleration signals were measured up to eight ensembles from Sensors 2 and 3, which are closely spaced as shown in Fig. 4.(Note that the choice of optimal sensor locations is a research issue that will be examined for a separate study.) The frequency-response-ratio function, FRR(2,3), defined as Eq. (3) was obtained from FRF(2) and FRF(3) measured at Sensors 2 and 3, as shown in Fig. 5. It is observed from the figure that both FRFs and FRRs were shifted due to the change in prestress levels. By setting the maximum prestress level, PS 1, as the undamaged baseline state, *FRRAC* values between the reference PS 1 and four other prestress levels (i.e., PS 2 - PS 4) were computed as shown in Fig. 6. The *FRRAC* values were decreased from the unity as the prestress-loss occurred(Note that we measured eight consecutive *FRRAC* values corresponding to eight ensembles of acceleration signals for each prestress level.). The damages were successfully alarmed for all prestress-loss cases.

3.2.2 Damage Estimation by Prestress-Loss Prediction Algorithm

In Step 2, the classified damage was estimated in

Table 2 Prestress-loss prediction in PSC girder

Prestress Level	Experiment		Prediction Results by 4 Modes				
	N(kN)	$\delta N/N$	Mode 1 $(\delta N/N)_1$	Mode 2 $(\delta N/N)_2$	Mode 3 $(\delta N/N)_2$	Mode 4 $(\delta N/N)_4$	Average $\delta N/N$
PS 1 (reference)	117.6	-	-	-	-	-	-
PS 2	98.0	0.17	0.12	0.21	0.17	0.18	0.17
PS 3	78.4	0.33	0.34	0.22	0.30	0.43	0.32
PS 4	58.8	0.50	0.50	0.28	0.47	0.50	0.44
PS 5	39.2	0.67	0.65	0.92	0.71	0.76	0.76

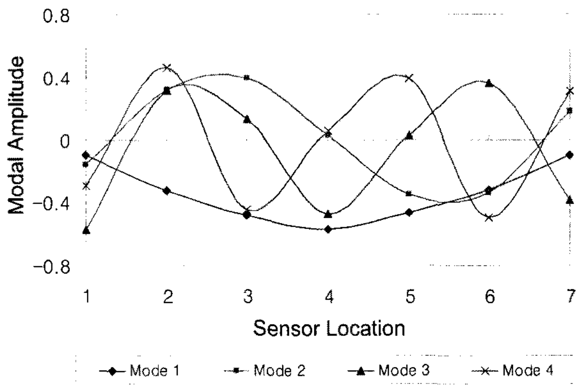


Fig. 7 Mode shapes extracted from acceleration signals

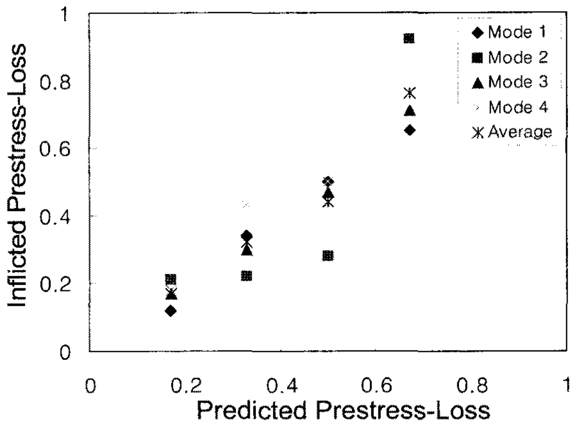


Fig. 8 Prestress-loss prediction in PSC girder

detail. Since the alarmed damage was identified as the prestress-loss, the frequency-based prestress prediction model(as described in Eq. (10)) was implemented to estimate the amount of the prestress-loss. The first four natural frequencies of the test structure were extracted from acceleration signals measured for the five prestress-loss levels, as summarized in Table 1. The corresponding mode shapes were measured as shown in Fig. 7. All six cases in Table 1 were examined to detect the prestress-loss and the results are outlined in Table 2.

The PS-loss prediction results show good matches with the inflicted ones. Prediction errors are as follows: 0%~30% for mode 1, even higher 24%~44% for mode 2, 0%~9% for mode 3, 0%~33% for mode 4, and 0%~14% for the average of all 4 modes. The predicted prestress-losses versus the inflicted prestress-losses were plotted in Fig. 8. The correlation between those two sets is relatively high and it means that the prestress-loss in the PSC girder can be detected via monitoring changes in natural frequencies.

4. Summary and Conclusion

In this paper, a vibration-based monitoring system was newly proposed to predict the loss of prestress forces in prestressed concrete(PSC) girder bridges. Firstly, a prestess-loss monitoring scheme was designed as follows: 1) To alarm the occurrence of prestess-loss in PSC girder by using the change in frequency response functions, 2) To estimate the extent of the alarmed damage by using a frequency-based prestress-loss prediction method. Finally, the feasibility of the proposed method was experimentally evaluated on a scaled PSC girder model for which acceleration responses were measured for several damage scenarios of prestress-loss.

Total five prestress-loss cases were simulated on the PSC girder. The occurrence of damage was alarmed by measuring the change in frequency responses from accelerometers mounted on the PSC girder. Once alarmed, the extent of damage was estimated with relatively good accuracy.

Acknowledgement

This study was supported by Korean Science and Engineering Foundation(KOSEF) through Smart Infra-Structure Technology Center(SISTeC) in the program year of 2005.

References

Adams, R.D., Cawley, P., Pye, C.J., Stone, B.J. (1978) A Vibration Technique for Non-destructively Assessing the Integrity of Structures, *Journal of Mechanical Engineering Science*, 20, pp. 93~100.

Bendat, J.S., Piersol, A.G. (1993), *Engineering applications of correlation and spectral analysis*, Wiley, USA.

Brinker, R., Zhang, L., Andersen, P. (2001) Modal Identification of output-only Systems using frequency Domain Decomposition, *Smart Materials and Structures*, 10, pp.441~445.

Doebling, S.W., Farrar, C.R., Prime, M.B. (1998) A summary review of vibration-based damage identification methods, *The Shock Vibration Digest*, 30(2), pp.91~105.

Kim, J.T., Ryu, Y.S., Cho, H.M., Stubbs, N. (2003a) Damage identification in beam-type structures: frequency-based method vs mode-shape-based method, *Engineering Structures*, 25, pp.57~67.

Kim, J.T., Yun, C.B., Ryu, Y.S., Cho, H.M. (2003b) Identification of prestress-loss in PSC beams using modal information, *Structural Engineering and Mechanics*, 17(3~4), pp.467~482.

Kim, J.T., Stubbs, N. (1995) Improved Damage identification Method based on Modal Information, *Journal of Sound and Vibration*, 259(1), pp. 57~67.

Law, S.S., Lu, J.R. (2005) Time domain responses of a prestressed beam and prestress identification, *Journal of Sound and Vibration*, 288(4~5), pp. 1011~1025.

Lin, T.Y. (1963) *Design of Prestressed Concrete Structures*, John Wiley & Sons, USA.

Miyamoto, A., Tei, K., Nakamura, H., Bull, J.W. (2000) Behavior of prestressed beam strengthened with external tendons, *Journal of Structural Engineering*, 126, pp.1033~1044.

Nawy, E.G. (1996) *Prestress Concrete - A Fundamental Approach*, Prentice Hall, USA.

Saiidi, M., Douglas, B., Feng, S. (1994) Prestress force effect on vibration frequency of concrete bridges, *Journal of Structure Engineering*, 120, pp.2233~2241.

Saiidi, M., Shield, J., O'connor, D., Jutchens, E. (1996) Variation of prestress force in a prestressed concrete bridge during the first 30 months, *PCI Journal*, 41, pp.66~72.

Stubbs, N., Osegueda, R. (1990) Global nondestructive damage evaluation in solids, *International Journal of Analytical and Experimental Modal Analysis*, 5(2), pp.67~79.

Yi, J. H., Yun, C. B. (2004) Comparative Study on Modal Identification Methods using Output-only Information, *Structural Engineering and Mechanics*, 17(3~4), pp.445~446.