Title

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Journal

Construction and Building Materials, 93

ISSN

0950-0618

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Publication Date

2015-09-01

DOI

10.1016/j.conbuildmat.2015.05.096

Peer reviewed

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ARTICLE *in* CONSTRUCTION AND BUILDING MATERIALS · SEPTEMBER 2015

Impact Factor: 2.27 · DOI: 10.1016/j.conbuildmat.2015.05.096

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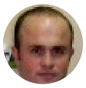

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Construction and Building Materials 93 (2015) 10–21

Contents lists available at ScienceDirect

Construction and Building Materials

journal homepage: www.elsevier.com/locate/conbuildmat

Finite element model calibration of precast structures using ambient vibrations

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HIGHLIGHTS

- The fem calibration of precast structures was investigated by AVT.
- The experimental measurements were carried out by OMA method.
- Analytical and experimental dynamic characteristics were compared.
- The initial fem were updated by changing stiffness coefficients of the joints.

ARTICLE INFO

Article history:
Received 21 July 2014
Received in revised form 13 April 2015
Accepted 2 May 2015

Keywords:
Precast structures
Operational Modal Analysis
Finite element model calibration
Semi-rigid connections
Ambient vibration test

ABSTRACT

In this paper, the finite element model calibration of precast structures was investigated by considering ambient vibration test results. Two precast structures, an overpass and a precast production facility, were selected for investigation. The initial finite element models of these structures were developed by using SAP2000 software and the initial dynamic characteristics were determined. The experimental measurements were carried out by Operational Modal Analysis method under ambient vibrations, such as wind and traffic loads, and the exact dynamic characteristics were identified experimentally. During the ambient vibration tests, structural responses were measured on different points of these precast structures. Measurement time, frequency span and effective mode number were determined by the pretest investigations. The modal parameters were extracted from the collected signals by Enhanced Frequency Domain Decomposition and Stochastic Subspace Identification techniques. At the end of the study, the analytical and experimental dynamic characteristics were compared with each other and the initial finite element models of these structures were updated by changing the stiffness coefficients of the connection joints.

1. Introduction

Finite element (FE) model calibration is a process that minimizes the differences between analytical and experimental results. The calibration process typically involves calibrating for static and dynamic conditions. The most preferred approach to calibrate initial analytical models is to use the modal parameters. This requires that experimental results of real structures be properly identified. The calibration process includes selection of calibrating parameters and performing optimization or manual analysis by trial and error.

In recent years, the using of precast structures is getting increase in the world [1,2]. Therefore, it is important to determine the theoretical dynamic behaviors of precast structures correctly in earthquake prone areas such as Turkey. The finite element models include many acceptances of structures from material properties to boundary conditions. If the finite element models of structures are constituted in light of the initial acceptances, it will be represented structures far from the truth. So, the initial finite element models ought to be calibrated in the light of dynamic characteristics obtained from experimental methods [3,4]. Some idealizations are made in the modeling of joints connections of precast structures. The joints of the elements are assumed to be ideally rigid or pinned. The assumptions on the joint connections affect the response of the precast structures considerably. It was presented that experimental results of eight 1/3 scale model precast concrete beam-to-column connections [5]. The test specimens consisted of

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http://dx.doi.org/10.1016/j.conbuildmat.2015.05.096
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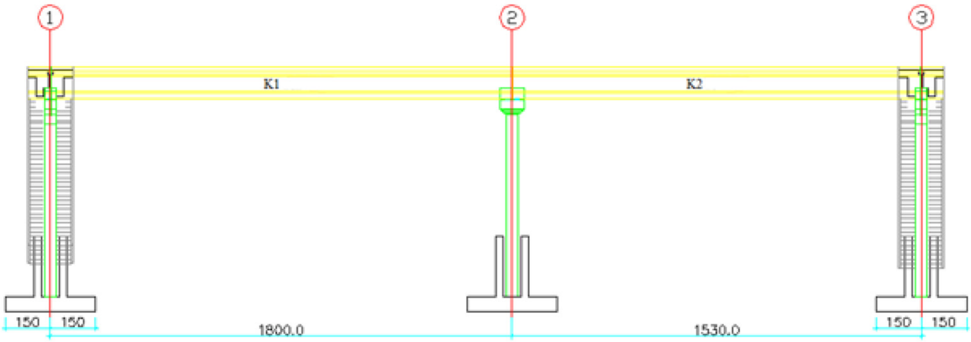
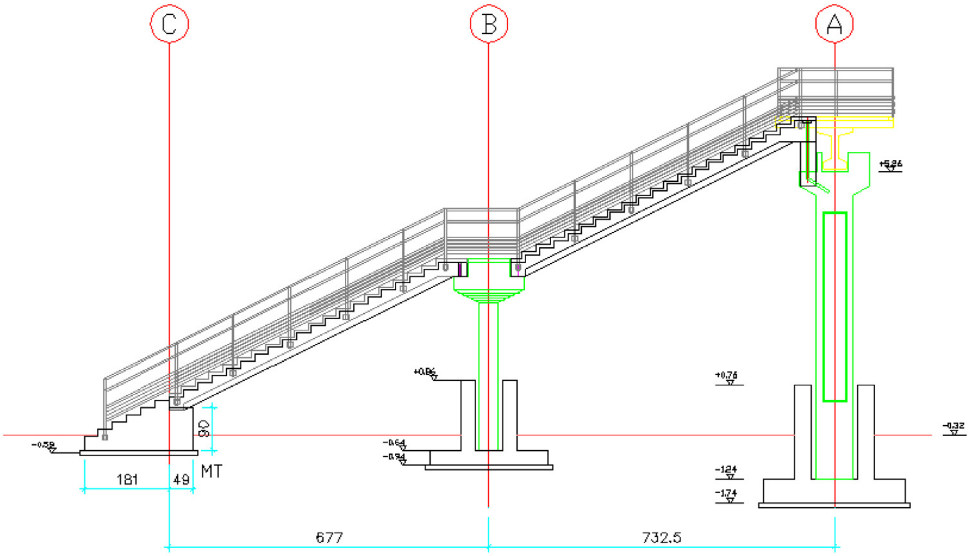


a. The longitudinal section of the C-1 Overpass.



b. The cross section of the C-1 Overpass.

Fig. 1. The dimensions of C-1 overpass.

Fig. 2. Some views from C-1 overpass.

interior connections designed in accordance with the 1985 Uniform Building Code provisions for Seismic Zones 2 and 4. The objective of the test program is to develop guidelines for an economical precast beam-to-column connection for regions of high seismicity. Park [6] described aspects of the design and construction of buildings in New Zealand incorporating precast concrete structural elements in floors, moment resisting frames, and structural walls. Design and construction for seismic resistance were also emphasized because that was where the greatest difficulties lie in the connection of precast concrete elements. Saqan [7] investigated different ductile beam-column connections for using in seismic-resistant precast frames. It was studied the responses of multistory flexibly connected frames subjected to earthquake excitations using a computer model [8]. The model incorporated connection flexibility as well as geometrical and material nonlinearities in the analyses. Connections were modeled as rotational springs with bilinear hysteretic moment–rotation relationships in their study. The study indicated that connection flexibility tends to increase upper stories interstory drifts but reduce base shears and base overturning moments for multistory frames. Öztürk and Çatal [9] investigated the dynamic response of semi-rigid frames by using a computer program. The connection flexibility was modeled by linear elastic rotational springs. Their study indicated that connection flexibility tends to increase vibration periods. López-Almansa et al. [10] studied on short to mid span-length light vaults made of reinforced brick masonry. Their research was oriented at proposing semi-prefabrication and construction techniques. Numerical models for predicting the structural behavior

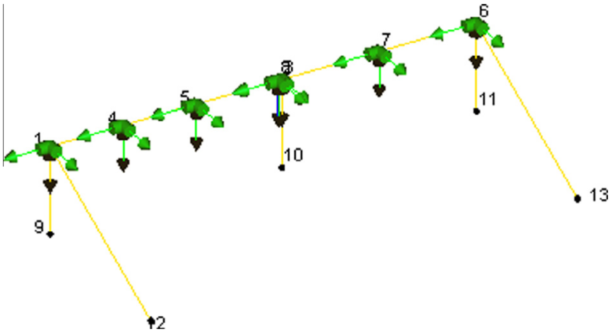


Fig. 3. The total measurement setup used in C-1 overpass.

of this type of structures were developed. Experimental research was performed with the following three main objectives: validating the proposed construction techniques, investigating the structural performance, and calibrating the numerical models.

The finite element model calibration of existing precast structures by using ambient vibration test results were aimed in this study. For this purpose, two precast structures were selected and they were modeled in SAP2000 finite element software [11]. It was carried out ambient vibration test of these precast structures and the natural frequencies, mod shapes and modal damping ratios were extracted from the measured responses. Experimental and analytical results were compared with each other. Initial analytical models were calibrated by considering the stiffness coefficients of joints according to the experimental results.

2. Ambient vibration test and modal parameter identification

Modal testing is a form of vibration testing that determines the natural frequencies, modal damping ratios and mode shapes of structures. A modal test procedure consists of an acquisition phase and an analysis phase. The method's common applications include not only estimation of dynamic characteristics but also damage detection and monitoring of structural performance. Experimental Modal Analysis (EMA) or Forced Vibration Testing and Operational Modal Analyses (OMA) or ambient vibration testing are used in the tests depending on the source of vibration [12].

In the OMA or ambient vibration testing, modal parameters are extracted from the measured responses. Two techniques are commonly used in modal parameter identification: Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification (SSI).

2.1. Description of Enhanced Frequency Domain Decomposition technique

In EFDD method the relationship between the unknown input and the measured responses can be written [13–15]:

$$G_{yy}(j\omega) = H(j\omega)^{*} G_{xx}(j\omega) H(j\omega)^{T} \tag{1}$$

where G_{xx} is the Power Spectral Density (PSD) matrix of the input signal, G_{yy} is the PSD matrix of the output signal, H is the Frequency Response Function (FRF) matrix, and * and T denote complex conjugate and transpose respectively. After some mathematical manipulations the output PSD can be reduced to a pole/residue form as follows:

$$[G_{yy}(\omega)] = \sum_{k=1}^{m} \left(\frac{[A_k]}{j\omega - \lambda_k} + \frac{[A_k]^{*}}{j\omega - \lambda_k^{*}} + \frac{[B_k]}{-j\omega - \lambda_k} + \frac{[B_k]^{*}}{-j\omega - \lambda_k^{*}} \right) \tag{2}$$

where A_k and B_k are the k-th residue matrices of the output PSD; λ_k is the pole [16].

2.2. Description of Stochastic Subspace Identification technique

The Stochastic Subspace Identification (SSI) technique is a time-domain method that directly works with time data, without the need to convert them to correlations or spectra. The Stochastic Subspace Identification algorithm identifies the state space matrices based on the measurements by using robust numerical techniques. Once the mathematical description of the

x-direction y-direction

z-direction

Fig. 4. The accelerometer connections for C-1 overpass.

Fig. 5. Data acquisition system and the connection cables.

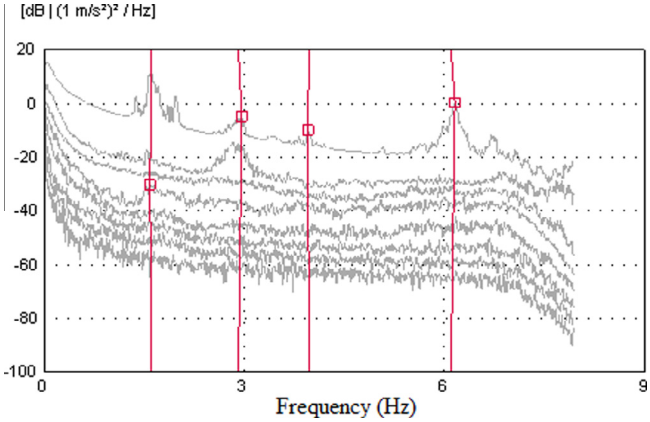


Fig. 6. The power spectral density function from EFDD technique.

structure (the state space model) is found, it is straightforward to determine the modal parameters.

The model of vibration structures can be defined by a set of linear, constant coefficient and second-order differential equations in the SSI technique [16,17]:

$$M\ddot{U}(t) + C_*\dot{U}(t) + KU(t) = F(t) = B_*u(t) \tag{3}$$

where M, C_*, K are the mass, damping and stiffness matrices, $F(t)$ is the excitation force, and $U(t)$ is the displacement vector at continuous time t. Observe that the force vector $F(t)$ is factorized into a matrix B_* describing the inputs in space and a vector $u(t)$. The detailed theoretical background is given in the literature [16,17].

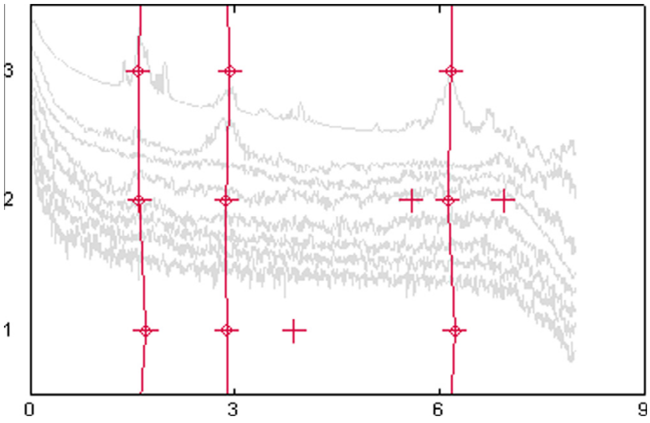


Fig. 7. The power spectral density function from SSI technique.

3. Applications

3.1. Overpass

The overpass was located on the Trabzon-Rize highway in Turkey. This overpass is made of prefabricated reinforced columns, beams and staircases. The total span of this overpass is 33.8 m, the width is 2.5 m and the height is 7 m. The geometry of this overpass is not symmetric; the left side is 18 m length and the right side is 15.8 m length. The dimensions of the overpass are shown Fig. 1. Some views from the overpass are given in Fig. 2.

3.1.1. Ambient vibration testing of the overpass

The Operational Modal Analysis (OMA) method was conducted on the overpass to determine its natural frequencies, mode shapes and modal damping ratios. The measurement using ambient vibrations was carried out three sub-steps with reference point. In this type of measurements, an accelerometer is kept as fixed and the other accelerometers are moved to the other set of measurement points. The total measurement setup and accelerometers directions are given in Fig. 3.

The uni-axial seismic accelerometers, which have 10 V/g sensitivity and 1–1500 Hz frequency range, were placed beginning, midpoint and ending points of each span using connection devices. Eight accelerometers were used for each measurement set and an additional accelerometer was also attained as reference on the midpoint of overpass span. The accelerometers were placed in x, $-y$ and z directions and the reference accelerometer was placed in the vertical direction. The accelerometers connections are given in Fig. 4.

B&K 3560 data acquisition system with 17 channels were used to transfer measured signals into the portable computer. The data acquisition system and measurement system are presented Fig. 5.

During the test, the frequency span was selected as 0–6.25 Hz considering the pre-test results. The measurements were performed during 45 min and the excitations were provided from environmental effects such as wind, traffic and pedestrian loads. Response signals obtained from the measurements were recorded and processed by OMA software [18]. At the end, the modal parameters were extracted from the analyzed signals by EFDD and SSI techniques. The power spectral densities matrices of the overpass obtained using the EFDD and SSI techniques are given in Figs. 6 and 7. The stabilization diagrams from SSI technique are also given in Figs. 8–10 for each step. The stabilization diagrams show the

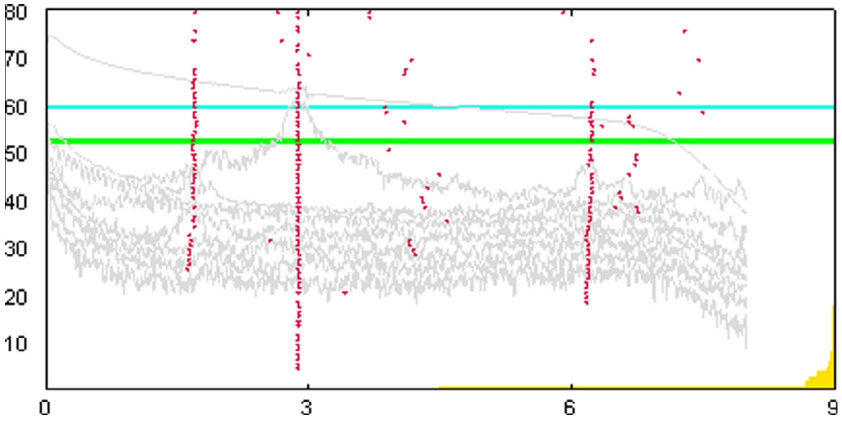


Fig. 8. The stabilization diagram of SSI technique for the first measurement step.

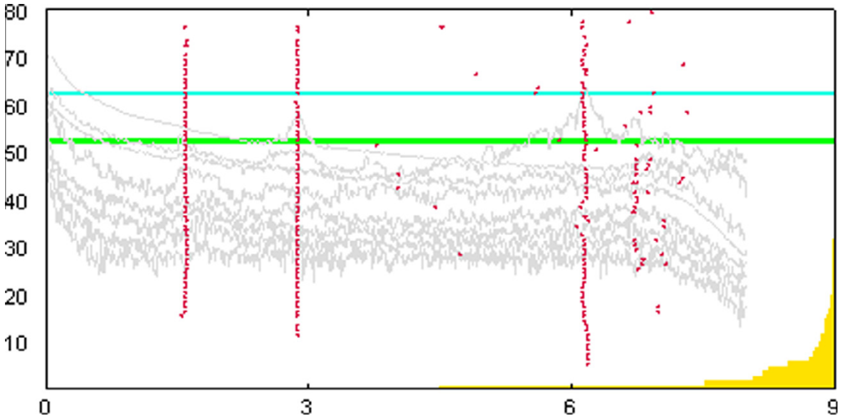


Fig. 9. The stabilization diagram of SSI technique for the second measurement step.

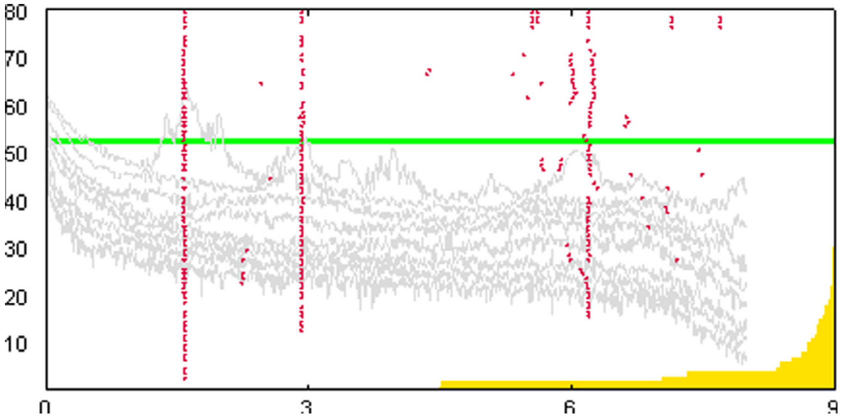


Fig. 10. The stabilization diagram of SSI technique for the third measurement step.

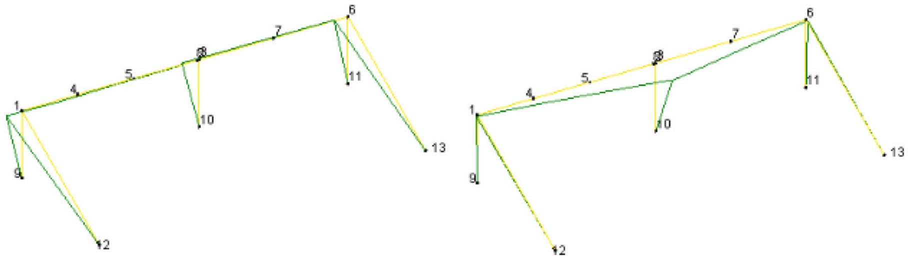
continuity of the resonance effects on the mode by the ambient vibrations.

The first three mode shapes of the overpass are extracted from these spectra and presented in Fig. 11. The first three modes of the overpass were the longitudinal direction, the transverse direction and vertical direction. The first three natural frequencies and corresponding modal damping ratios attained by EFDD and SSI methods are given in Table 1.

The natural frequencies of the overpass attained by the EFDD and SSI techniques are very close to each other. Because of the nearly symmetric shape of the overpass, the first two bending modes have close frequency values. The first three vibration modes are identified from ambient vibration data in the frequency range of 1.5–6.5 Hz. The average modal damping ratios of the overpass are 0.84% and 1.734% for EFDD and SSI techniques, respectively.

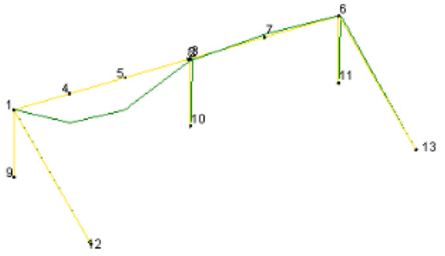
3.1.2. Finite element model of the overpass

To validate the identified modal parameters, FE analysis is performed. Initial FE model is developed using SAP2000 software [11] and the modal analysis is carried out. The overpass is modeled by



First Mode (longitudinal)

Second Mode (transverse)



Third Mode (transverse)

Fig. 11. The mode shapes of the C-1 overpass from the measurement setup.

Table 1
The experimental natural frequencies and modal damping ratios of C-1 overpass.

Mode number	EFDD technique		SSI technique	
	Frequencies (Hz)	Modal damping ratios (%)	Frequencies (Hz)	Modal damping ratios (%)
1	1.604	0.347	1.627	2.203
2	2.917	1.684	2.895	1.44
3	6.104	0.489	6.178	1.559

Table 2
Material properties of the overpass.

Materials	Elasticity modulus (N/m^2)	Poisson ratios	Mass Densities (kg/m^3)
Columns (C40)	3.4×10^{10}	0.2	2300
Prestressed beams (C40)	3.4×10^{10}	0.2	2300
Staircases (C20)	2.8×10^{10}	0.2	2300
Columns for staircases (C40)	3.4×10^{10}	0.2	2300
Landings (C20)	2.8×10^{10}	0.2	2300
Aprons (C20)	2.8×10^{10}	0.2	2300

using beam elements having six degree of freedoms. The overpass's apron is made of two precast reinforced beams. In the initial model, it is made some idealizations that the all semi-rigid connections have same stiffness coefficient which is 10,000 kNm/rad on the beam-to-column joints. Also, the material properties are taken from its project. The elasticity modulus, mass densities and Poisson ratios of the precast members are taken in Table 2. Fig. 12 shows the developed initial finite element model of the overpass.

The modal analysis of the initial finite element model was carried out and the first three natural frequencies and their corresponding mode shapes were determined. The natural frequencies of the initial analytical model are given in Table 3. Fig. 13 demonstrates the first three analytical mode shapes of the overpass's apron.

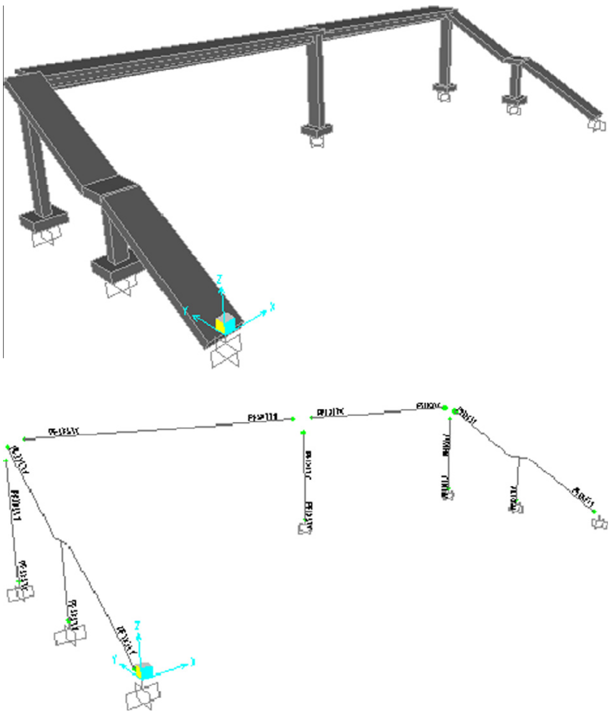


Fig. 12. The initial finite element model of the C-1 overpass.

3.1.3. Calibration of the overpass

The analytical and experimental investigations were compared by considering the natural frequencies and mode shapes. The first three natural frequencies of the overpass are compared in Table 4 for the analytical and the experimental investigations.

As it can be seen in Table 4, differences between experimental and analytical results are very big. Because of this condition, it will be made a calibration operation to determine the real behavior of structure.

Table 3
The analytical natural frequencies and periods of the overpass.

Mode	Frequencies (Hz)	Periods (s)	Modal behavior
1	2.284	0.438	Longitudinal mode
2	3.411	0.293	Transverse mode
3	4.536	0.22	Transverse mode

It is thought that the material and section properties are not changed. Connection rigidities on the joints were considered as the calibration parameter. The model calibration of the precast reinforced concrete model was done manually. It was aimed to minimize the difference between analytical and experimental natural frequencies by changing the percentage of connection rigidity. Connection rigidities are determined between 10,000 and 50,000 kNm/rad. Therefore, percentages of the differences between experimental and analytical frequencies were reduced depending on previous experiences. Experimental and calibrated frequencies which were obtained at the end of the calibration process are given in Table 5.

3.2. Precast production facility

The precast production facility was located in Trabzon-Esiroğlu, Turkey. The facility was made of prefabricated reinforced columns, beams, purlins and roof trusses. The distance of between each span of frame is 7.5 m and the total span of the facility is 75 m, the width is 20.05 m and the height is 10.516 m. The eleven roof trusses were used and for each span fourteen purlins were used. The distance of between each purlin is 1.55 m. The dimensions of the facility are shown Fig. 14. The some views from the precast production facility are given in Fig. 15.

Table 4
The analytical and experimental natural frequencies of C-1 overpass.

Mode numbers	Natural frequencies (Hz)				
	Experimental		Analytical	Differences (%)	
	EFDD	SSI		EFDD	SSI
1	1.604	1.627	2.284	42.4	40.38
2	2.917	2.895	3.411	16.9	17.82
3	6.104	6.178	4.536	25.69	26.58

Table 5
The experimental and calibrated natural frequencies of the C-1 overpass.

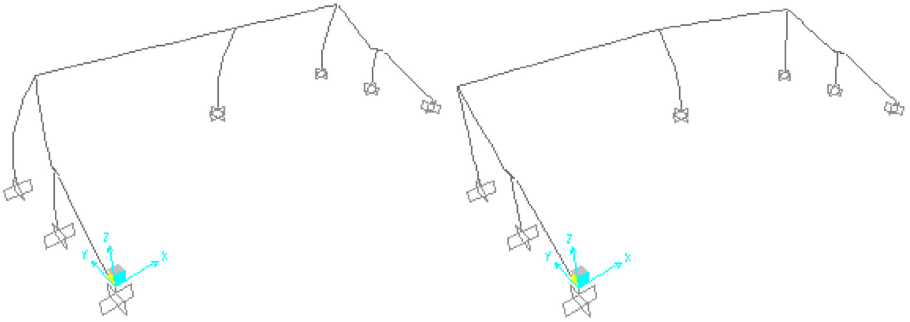
Mode numbers	Natural frequencies (Hz)					
	Experimental		Analytical		Differences (%)	
	EFDD	SSI	Before calibration	After calibration	EFDD	SSI
1	1.604	1.627	2.284	1.612	0.5	0.92
2	2.917	2.895	3.411	2.91	0.24	0.52
3	6.104	6.178	4.536	6.051	0.87	2.00

3.2.1. Ambient vibration testing of the precast production facility

To determine the precast production facility's natural frequencies, mode shapes and modal damping ratios, the OMA method was used. The measurement was carried out one time without reference point. The seismic accelerometers which have 10 V/g sensitivity were placed beginning, midpoint and ending points of frame using devices because of inadequate accelerometer numbers. Totally, twelve accelerometers which were located in x and y directions were used for the measurement. The accelerometers connections are given in Fig. 16. The total measurement setup and accelerometers directions are given in Fig. 17.

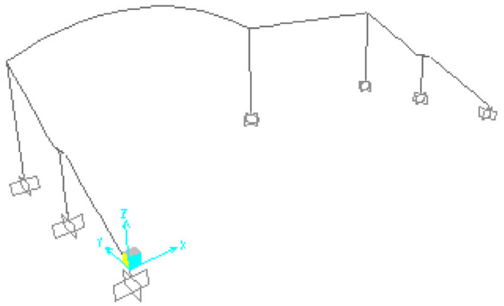
Also B&K 3560 data acquisition system with 17 channels were used to transfer measured signals into the portable computer. The data acquisition system and measurement system are presented Fig. 18.

During the test the frequency span was selected as 0–6.25 Hz. The measurements were performed during 30 min and the



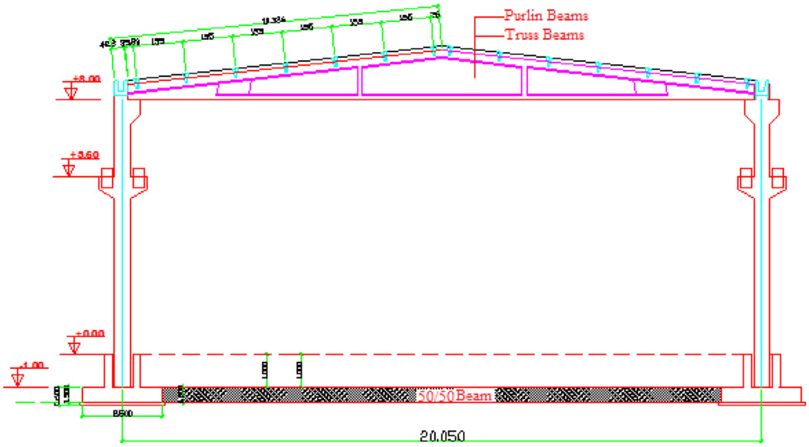
First Mode (longitudinal)

Second Mode (transverse)

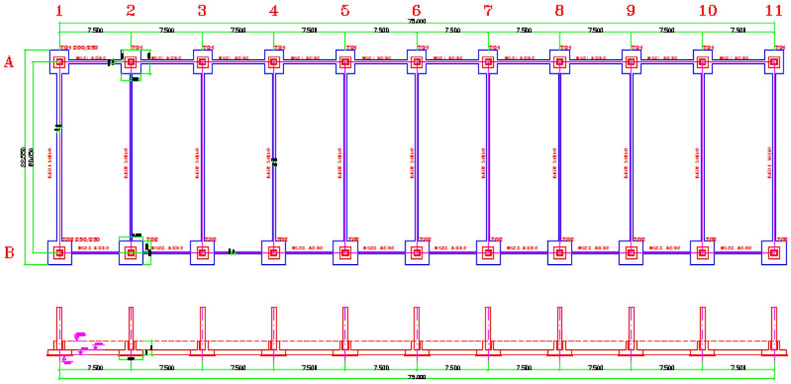


Third Mode (transverse)

Fig. 13. The first three mode shapes of the overpass from the initial FEM.



a. Cross section of facility.



b. Foundation plan of facility.

Fig. 14. The dimensions of Doğu Karadeniz precast production facility.

Fig. 15. Some views from precast production facility.

x-direction y-direction

Fig. 16. The accelerometer connections for the precast production facility.

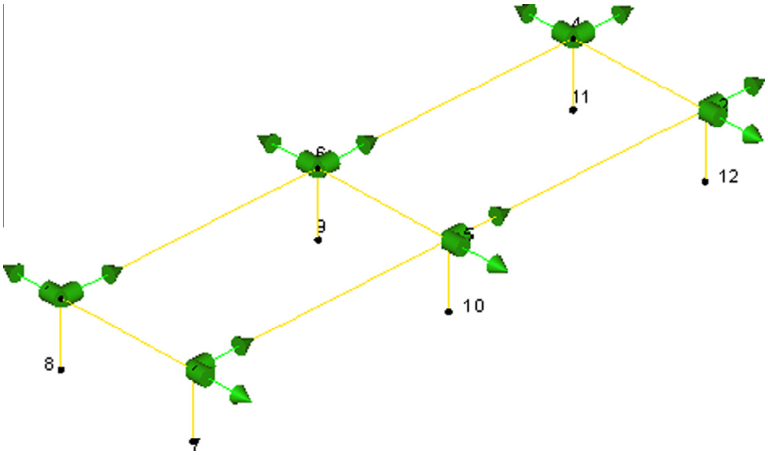


Fig. 17. The total measurement setup used in the precast production facility.

Fig. 18. Data acquisition system and the connection cables.

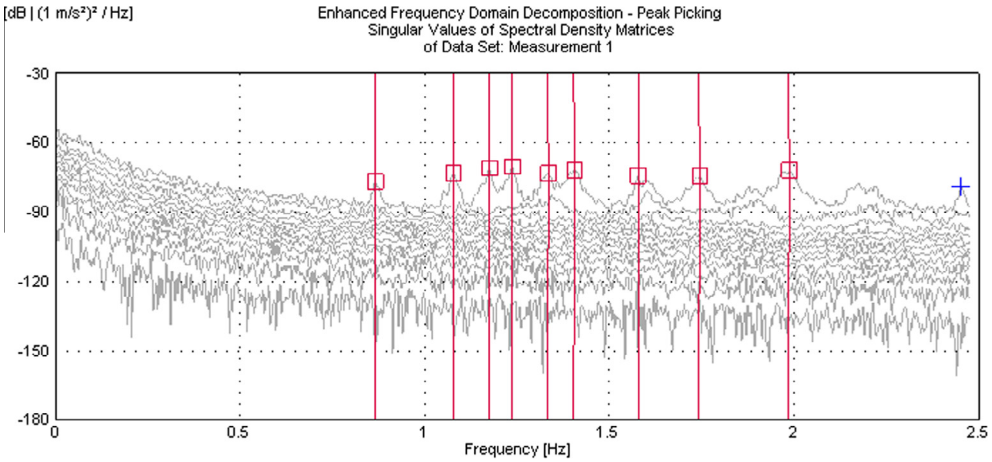


Fig. 19. The power spectral density function from EFDD technique.

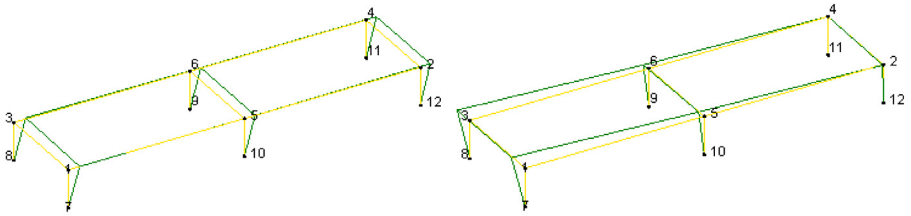
excitations were provided from environmental effects such as wind and traffic loads. Response signals obtained from the measurements were recorded and processed by OMA software [18]. At the end, the modal parameters were extracted from the analyzed signals by EFDD. The power spectral densities matrices of the precast production facility obtained using the EFDD technique are given in Fig. 19.

The first five mode shapes of the precast production facility are extracted from this spectral function and presented in Fig. 20. The first mode of the precast production facility was the lateral longitudinal direction and all the other modes were the lateral transverse directions. The first five natural frequencies and corresponding modal damping ratios attained by EFDD are given in Table 6.

The first five vibration modes are identified from ambient vibration data in the frequency range of 0.8–1.4 Hz. The average modal damping ratio of the precast production facility is 0.708% for EFDD technique.

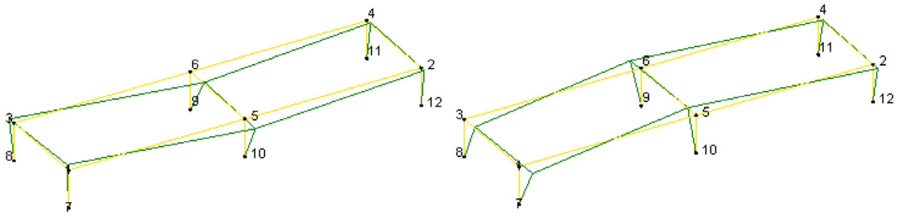
3.2.2. Finite element model of the precast production facility

To confirm the identified modal parameters, FE analysis was performed. Initial FE model was developed using SAP2000



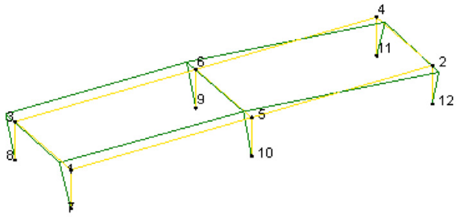
First Mode

Second Mode



Third Mode

Fourth Mode



Fifth Mode

Fig. 20. The mode shapes of the precast production facility from the experimental measurement.

Table 6
The experimental natural frequencies and modal damping ratios of the precast production facility.

Mode numbers	EFDD technique	
	Frequencies (Hz)	Modal damping ratios (%)
1	0.869	0.774
2	1.079	0.841
3	1.176	0.428
4	1.240	0.772
5	1.338	0.724

Table 7
Material properties of the precast production facility.

Materials	Modulus of elasticity (N/m^2)	Poisson ratios	Mass densities (kg/m^3)
Frame (C30)	3.2E10	0.2	2300
Foundation (C20)	2.8E10	0.2	2300

software and the modal analysis was carried out. The precast production facility was modeled by using beam elements having six degree of freedoms. 454 frames and 110 areas were used to model the structure. Stiffness coefficient values assigned the lower ends of columns were used more than another joints. Fixed supports were assigned the lower ends of columns. In the initial models, it was made some idealizations that the all semi-rigid connections had same stiffness coefficient which is 10,000 kNm/rad on the beam-to-column joints. The material and section properties were taken from its architectural project. The elasticity modulus, Poisson's ratios and mass densities of the precast members were taken in Table 7. Fig. 21 shows the developed initial finite element model of the precast production facility.

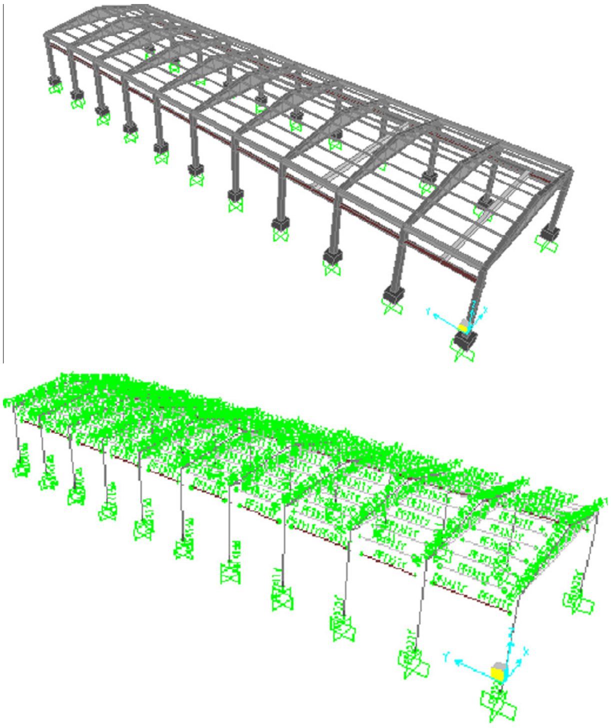


Fig. 21. The initial finite element model of Doğu Karadeniz precast production facility.

The modal analysis of the initial finite element model was carried out and the first five natural frequencies and their corresponding mode shapes were determined. The natural frequencies of the initial analytical model are given in Table 8. Fig. 22 demonstrates

Table 8
The analytical natural frequencies and periods of the precast production facility.

Mode	Frequencies (Hz)	Periods (s)	Modal behavior
1	0.861	1.148	Lateral mode
2	0.918	1.089	Torsion mode
3	1.001	0.999	Longitudinal mode
4	1.069	0.936	Torsion mode
5	1.270	0.787	Torsion mode

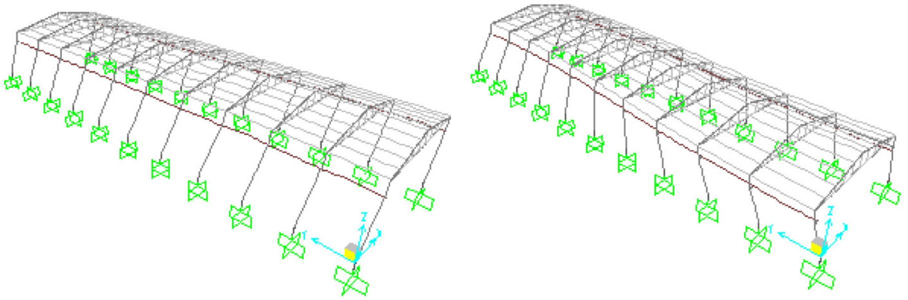
the first five analytical mode shapes of the precast production facility.

3.2.3. Calibration of the precast production facility

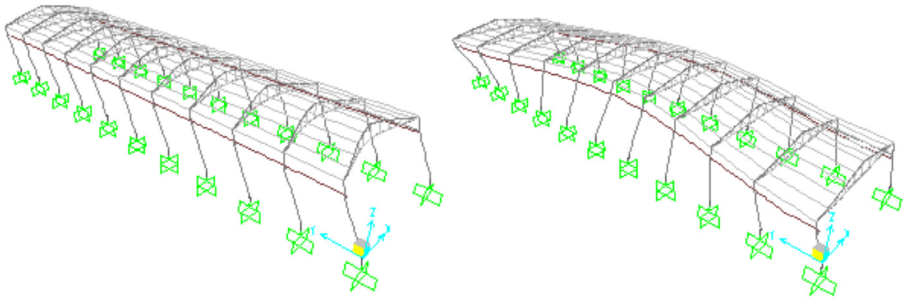
The analytical and experimental parameters were compared with each other by considering the natural frequencies and mode shapes. The first five natural frequencies and differences of the precast production facility are given in Table 9 for the analytical and the experimental investigations. The results from the EFDD technique are considered as the experimental values.

As can be seen in Table 9, the biggest difference between experimental and analytical results is about 15%. This difference is a big value. Because of this condition, it was made a calibration operation to determine the real behavior of structure and to create model close to reality.

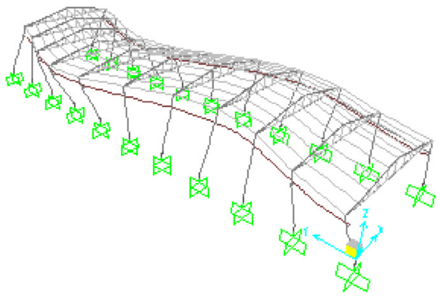
While the FE model is created using SAP2000 software, it is made some idealizations that the all connections have same rigidity on base and connections joints. Because the material properties are taken from its architectural project, material properties were not considered as calibrating parameters. Connection rigidities on the joints were considered as the calibration parameter. In this study, the model calibrations of the precast reinforced concrete models are done manually. To minimize the difference between analytical and experimental natural frequencies, the percentage of connection rigidity of the joints is changed. Connection rigidities are determined between 10,000 and 30,000 kNm/rad. Therefore, percentages of the differences between experimental and analytical frequencies were reduced depending on previous experiences. Experimental and calibrated frequencies which are obtained at the end of the calibration process are given in Table 10.



First Mode Second Mode



Third Mode Fourth Mode



Fifth Mode

Fig. 22. The first five mode shapes of the precast production facility from the initial FEM.

Table 9
The analytical and experimental natural frequencies of the precast production facility.

Mode numbers	Natural frequencies (Hz)		
	Experimental (EFDD)	Analytical	Differences (%) (EFDD)
1	0.869	0.870	0.00
2	1.079	0.918	14.92
3	1.176	1.001	14.88
4	1.240	1.069	13.79
5	1.338	1.270	5.08

Table 10
The experimental and calibrated natural frequencies of Doğu Karadeniz precast production facility.

Mode numbers	Natural frequencies (Hz)			
	Experimental (EFDD)	Analytical		Differences (%) (EFDD)
		Before calibration	After calibration	
1	0.869	0.870	0.860	0.00
2	1.079	0.918	1.078	0.00
3	1.176	1.001	1.119	4.84
4	1.240	1.069	1.240	0.00
5	1.338	1.270	1.364	1.94

4. Conclusions

The effects of model calibration on the dynamic characteristics of two precast structures were investigated in this study. The study consists of experimental measurements, analytical modeling and FE model calibrations. The ambient vibration testing were performed under environmental loads. The results drawn from this study are given below:

The first three experimental vibration modes within the frequency range 1.5–6.5 Hz were successfully identified using the EFDD and SSI techniques for the overpass. An excellent agreement was found between the modal estimates obtained from the two experimental methods. The first three modes of the overpass were the longitudinal direction, the transverse direction and transverse direction.

The first five vibration modes within the frequency range 0.8–1.4 Hz were successfully identified using the EFDD technique for the precast production facility. The average modal damping ratio of the overpass was obtained as 0.708%. The first mode of the precast production facility was the lateral longitudinal direction and all the other modes were the lateral transverse directions.

The analytical frequencies of the initial finite element models of the overpass and the precast production facility were obtained within the range 2.280–4.54 Hz and 0.87–1.27 Hz, respectively. The developed initial FE model turned out to the moderate agreement between the mode shapes but some differences exist between analytical and experimental frequencies. The biggest difference between experimental and analytical results is about 43% for the overpass and about 15% for precast production facility. Because of these big differences, it was made a calibration operation to determine the real behavior of structure for each structure. In the initial FE models, it is made some idealizations that all semi-rigid connections have same coefficients. It is thought that the connections on the precast structures have not got the same stiffness coefficient; therefore each connection was assigned different stiffness coefficient values in the calibrated FE models. After the calibrations, the biggest differences between the analytical and experimental frequencies decrease 2.0% for the overpass and 4.84% for the precast production facility.

The study clearly emphasize that the static and dynamic behaviors of precast structures should be determined and evaluated considering the calibrated finite element models for the safety of these structures. The calibrated models can be used in the earthquake analysis of these structures.

Acknowledgements

This research was supported by the TUBITAK and Karadeniz Technical University under Research Grant No. 106M038 and 2005.112.001.1, respectively. I also thanks to Emre SARI, Fatma Nur Turan, Işık EYÜBOĞLU and 10th Regional Directorate of Highways.

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