SEISMIC PERFORMANCE ASSESSMENT OF GOLDEN HORN METRO BRIDGE THROUGH STRUCTURAL HEALTH MONITORING SYSTEM

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ABSTRACT:

In this study, seismic performance of Golden Horn Metro Bridge is assessed using the existing health monitoring system. A study is currently underway to be able to utilize the data obtained from the structural health monitoring system for seismic performance assessment of the Golden Horn Bridge using either directly time-history analysis or fragility curves after an earthquake. Main parts of this study are (a) development of software that can estimate natural vibration periods, mode shapes and the damping ratios from the data obtained from the structural health monitoring system, (b) updating the mathematical model using this information, and (c) performance and damage assessment of the bridge after a real or scenario earthquake event. For this purpose, a software is developed that can estimate the bridge structure properties using the acceleration data obtained from the bridge using peak picking, frequency domain decomposition and stochastic subspace identification method. The program is verified first by vibration experiments that are conducted on a simply supported steel beam at the ITU Steel Lab and second by comparing the results of the analysis of the bridge data with the results of a commercial system identification software package. After the verification process, the modal properties of the bridge are obtained using the data collected at various days and times. A nonlinear mathematical model of the bridge comprised of frame elements and rigid-plastic hinges is established using the bridge design calculation reports and drawings. This model is updated according to results of the system identification study. Performance assessment of the bridge is performed using nonlinear time-history analysis of the updated model and historical ground motion data that are scaled to various levels of seismicity, which are used in the design of the bridge. The results show that the bridge satisfies the design performance goals. Further a preliminary fragility study is provided by investigating the relation between the damage in the critical elements and the seismic hazard levels. In overall, the health monitoring system can be used to find the in-situ structural properties and structural damage in the bridge after an earthquake.

KEYWORDS: structural health monitoring, system identification, modal analysis, finite elements, model updating, earthquake, Golden Horn Metro Bridge

1. INTRODUCTION

System identification is widely used for structural behavior estimation from field measurements. Field measurements provide a reference point for comparing real physical model and mathematical model of structures for future changes. It is also used for matching the mathematical model to physical model. Another related purpose of system identification studies is to determine whether the structure is damaged and identify the possible locations of the damage. After the damage is located, the damage can be repaired possible without loss of functionality. For certain structures, remaining service life of the structure can also be estimated and necessary executive decisions can be made. The structural properties can be measured using system identification methods throughout the lifetime of the structure, which collectively is called *structural health monitoring*.

Structural health monitoring and the system identification methods can serve as tools to assess important structures before and after natural hazards such as earthquakes. Current practice in civil/structural engineering is simply to use system identification to estimate structure vibration frequencies and mode shapes after the hazard and compare it to the values estimated from the measurements taken before the hazard. While there is significant research in identification of the damage locations, it is not reported that these methods are effectively used in in-situ structures. In general, there is a need to assess how effective structural health monitoring methods for full-scale in-situ structures after major hazards, particularly earthquakes. There is also a need to understand how the health monitoring system should be established considering the specific design approaches of the structure. In this study, seismic use of structural health monitoring system of Golden Horn Metro Bridge (GHMB) is investigated to assess effectiveness and possible use of such systems.

The GHMB is for metro crossing of Haciosman-Yenikapi M2 metro line. The construction of the bridge is started in 2009, and the bridge is opened in 2014. The GHMB is used for metro-crossing purposes belonging to M2 metro line and operated by Metro Istanbul. Average number of passenger using this metro line is approximately 320.000, according to *M2 Metro Line information*. The Golden Horn Bridge consists of five separate bridges: cable-stayed bridge, swing bridge, single-span bridge, Beyoglu approach bridge, and Unkapani approach bridge. Beyoglu and Unkapani approach bridges are reinforced concrete bridges, while other bridges are constructed as steel structures. The bridge is designed by Wiecon Consulting Engineers (WCE), and there is a permanent health monitoring system on the bridge, which is established by Vienna Consulting Engineers (VCE). The permanent bridge health monitoring system includes sensors for global positioning system (GPS), temperature, displacement, slope and acceleration measurement. The data measured by the sensors are collected at the Metro Istanbul Headquarters. In addition, a temporary health monitoring system is established for initial measurements after the construction of the bridge Furtner *et al.* (2019). Based on the first measurements, structural properties are estimated and compared with the properties used in the design. The results are also used to calibrate the permanent health monitoring system. However, there is no available software for system identification of the bridge.

Figure 1. General view of GHMB.

Upon discussion with the operating agency, Metro Istanbul, a study is taken underway to setup a system for system identification considering earthquakes. The study also aims to understand the structural and seismic design philosophy and how the health monitoring system can be integrated into this philosophy. For this purpose, a MATLAB-based software tool is developed for the system identification of GHMB using several system identification algorithms. The selected algorithms are peak picking, frequency domain decomposition and stochastic subspace identification. To verify the software and the algorithms, they are used to identify the structural properties of a simple vibrating steel beam from the measurements taken in the laboratory environment . After verification of the software developed, algorithms are applied to the GHMB using the field-measurements, and the results are presented. Finally, a simple fragility curve study is conducted as an example to demonstrate how the health monitoring system can be used after a major earthquake to make a decision on the damage state of the bridge.

2. THEORETICAL BACKGROUND

In this section, the system identification methods used are explained briefly. These methods are peak picking, frequency domain decomposition and stochastic subspace identification. Half power bandwidth method, which is used for damping estimation in the frequency domain is not explained herein.

2.1. Peak Picking

Peak Picking technique is also called Basic Frequency Domain (BFD) method. This method is broadly defined in the classical reference book Bendat and Piersol (1980) and Felber (1993). This is the one of the easiest method to implement in the class of output-only modal parameter identification techniques. It is generally based on Power Spectral Density (PSD) of the sensor measurements. This method is considers the generalized a SDOF system of a mode in operational modal analysis (OMA) and based on approaches applicable to SDOF systems. For the implementation of BFD method, structural modes of interest should be excited, and structure should be in the linear region. Likewise, modes of interest should be well separated and lightly damped according to Rainieri and Fabbrocino (2014) and Felber (1993). This method is based on the assumption that the structural response can be proposed as approximately equal to modal response, while one mode is dominant in a wide frequency band. Brief equations related to the response of the structure can be represented as follows:

$$\mathbf{y}(t) = \mathbf{a}q(t), \tag{1}$$

$$\mathbf{R}(\tau) = E[y(t)y(t+\tau)^T] = \mathbf{a}E[q(t)q(t+\tau)]\mathbf{a}^{\mathrm{T}} = R_q(\tau)\mathbf{aa}^{\mathrm{T}}, \tag{2}$$

$$\mathbf{G}_{\mathrm{y}}(f) = \mathbf{G}_{\mathrm{q}}(f)aa^T. \tag{3}$$

Herein, $\mathbf{R}$ is the correlation function matrix of the responses, $\mathbf{G}_{\mathrm{y}}$ is the spectral density matrix of the response and $\mathbf{G}_{\mathrm{q}}$ is the auto spectral density matrix of modal responses.

2.2. *Frequency Domain Decomposition*

The frequency-domain decomposition (FDD) method is first proposed by Brincker *et al.* (2001), and it is a widely used for the output-only system identification of civil structures. This method is based on singular value decomposition to obtain the peak of the PSD matrix through singular value decomposition. The FDD method has the significant advantage on identification on closely-spaced modes (Brincker and Ventura 2015). Brief summary of the equations used are:

$$\mathbf{y}(t) = a_1q_1(t) + a_2q_2(t) + a_3q_3(t) + \ldots = \mathbf{A}q(t) \tag{4}$$

$$\mathbf{R}_{\mathrm{y}}(\tau) = E[y(t)y^T(t+\tau)] \qquad \mathbf{R}_{\mathrm{y}}(\tau) = \mathbf{A}E[q(t)q^T(t+\tau)]\mathbf{A}^{\mathrm{T}} \tag{5}$$

$$\mathbf{G}_{\mathrm{y}}(f) = \mathbf{A}\mathbf{G}_{\mathrm{q}}(f)\mathbf{A}^{\mathrm{H}} = \mathbf{A}[g_n^2(f)]\mathbf{A}^{\mathrm{H}} = \mathbf{USU}^{\mathrm{H}} = \mathbf{U}[s_n^2]\mathbf{U}^{\mathrm{H}} \tag{6}$$

2.3. *Data-Driven Stochastic Subspace Identification*

Stochastic subspace identification (SSI) methods are time-domain system identification methods. Time-domain system identification methods generally starts from the state-space representation of a linear time invariant system. The SSI methods are explained in detailed in Overschee and Moor (1996) and Peeters (2000). In the SSI methods, the first step is to construct the block Hankel matrix from measurement data directly. The block Hankel matrix obtained is used to compute the projection of the row space of the future inputs into row space of the past sensors through QR factorization. The projection $\mathscr{P}_i$ can be factorized observability matrix O_i and Kalman states $\hat{\mathbf{X}}_i$. Then, the state sequences $\hat{\mathbf{X}}_{i+1}$ can be computed using extended observability matrix $\mathbf{O}_{i-1}$ obtained by erasing the last l rows of $\mathbf{O}_i$ and the projection matrix $\mathscr{P}_{i-1}$. System matrix $\mathbf{A}$ and input influence matrix $\mathbf{C}$ is obtained using least square estimation. Finally, the matrices $\mathbf{A}$ and $\mathbf{C}$ can be used to mode frequency, mode shapes, and modal damping. The equations associated with this method are briefly given as:

$$\begin{aligned} \dot{x}(t) &= \mathbf{A}_c x(t) + \mathbf{B}_c u(t) \\ y(t) &= \mathbf{C}_c x(t) + \mathbf{D}_c u(t) \end{aligned} \tag{7}$$

$$H = \begin{pmatrix} \mathbf{Y}_p \\ \hline \mathbf{Y}_f \end{pmatrix}, \qquad \mathscr{P}_{i-1} = \mathbf{Y}_f^- / \mathbf{Y}_p^+ = \mathbf{O}_{i-1}\hat{\mathbf{X}}_{i+1}, \qquad \hat{\mathbf{X}}_{i+1} = \mathbf{O}_{i-1}^{\dagger}\mathscr{P}_{i-1}, \tag{8}$$

$$\begin{pmatrix} \hat{\mathbf{X}}_{i+1} \\ \mathbf{Y}_{i|i} \end{pmatrix} = \begin{pmatrix} \mathbf{A} \\ \mathbf{C} \end{pmatrix} \hat{\mathbf{X}}_i + \begin{pmatrix} \mathbf{W}_i \\ \mathbf{V}_i \end{pmatrix} \qquad \mathbf{A} = \mathbf{\Psi}\mathbf{\Lambda}_{\mathrm{d}}\mathbf{\Psi}, \qquad \mathbf{U} = \mathbf{C}\mathbf{\Psi}, \qquad \xi_i = \frac{Re(\lambda_i)}{|\lambda_i|}. \tag{9}$$

3. EXPERIMENTAL VALIDATION

In this part, the aforementioned system identification algorithms are developed and verified for an simple steel beam before applying them to the GHMB. The experimental study is explained in this section.

3.1. *Experimental Setup*

Experiments are performed using steel beam with a rectangular (steel) section with dimensions of 2.06 cm × 10 cm and a length of 242 cm (Figure 2). The boundary conditions of beam are set as roller support at one end and pin support at the other. Six acceleration sensor is used. After the impulse loading are applied to the beam, free decay responses of the beam's vibration are recorded. The selected free decay are analyzed, and the modal properties of the beam are obtained.

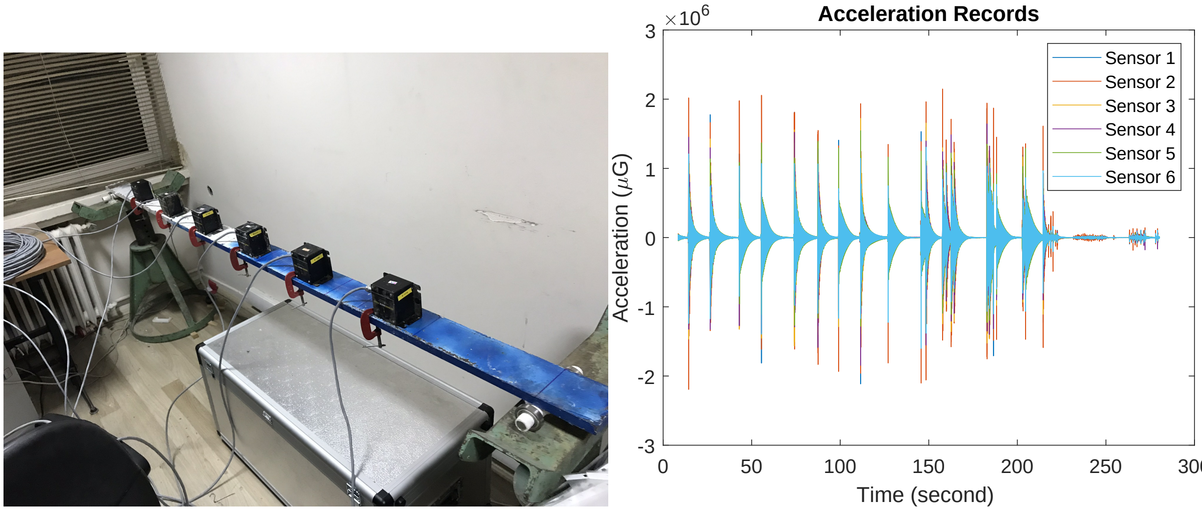

Figure 2. Experimental setup and acceleration records

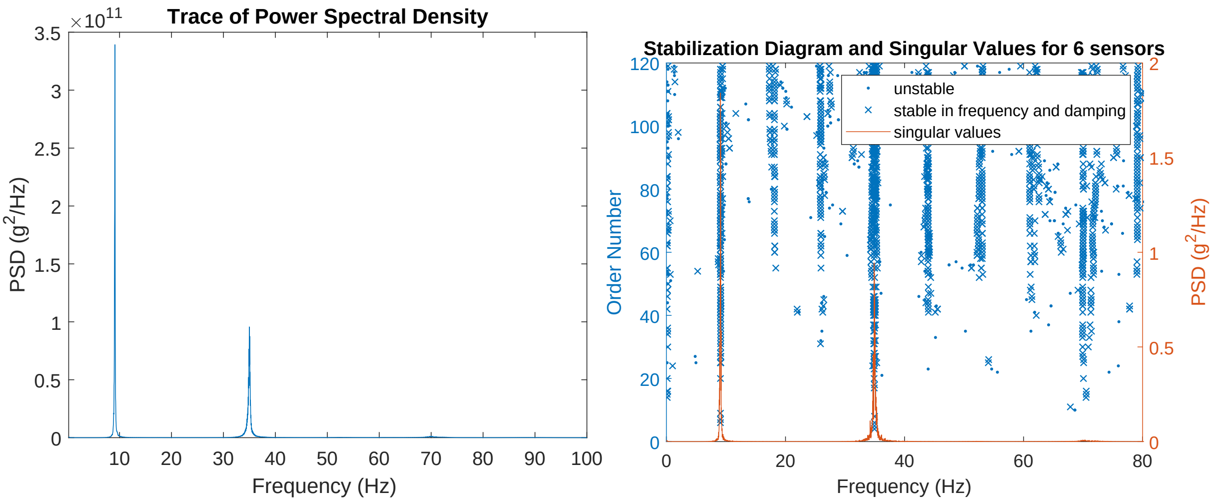

Figure 3. Trace of PSD and stabilization diagram of acceleration measurements

3.2. System Identification

BFD, FDD, and SSI algorithms are applied to the steel beam. Modal frequencies are compared in Figure 3. The the trace of the PSD shows the frequencies for the BFD method. Frequencies obtained from the FDD method (singular values) and SSI method (stable values) are shown in the second plot. The mode shapes are also obtained (not shown herein) with modal assurance criteria (MAC) values close to 99% for the first three modes compared to the mathematical model.

4. SYSTEM IDENTIFICATION OF GHMB

This section provides general information about the bridge and the field acceleration measurements.

4.1. Structural Properties of the Bridge

The GHMB bridge involves five major parts: main cable-stayed bridge, two approach bridges (Unkapani and Beyoglu), swing bridge, single-span bridge. The swing bridge, cable-stayed bridge, and single-span bridge is steel structures and approach bridges are concrete structures. This study focuses on the cable-stayed bridge. The span configurations of cable-stayed bridge is 90 m +180 m + 90 m. Structural components of the cable-stayed bridge are the pylons, deck, piers, and cables. The bridge deck consists of 3 cells. The width of deck is 13.7 m, its height is 3.5 m and its thickness is 40 mm. It is also approximately 17 m above water level. It is

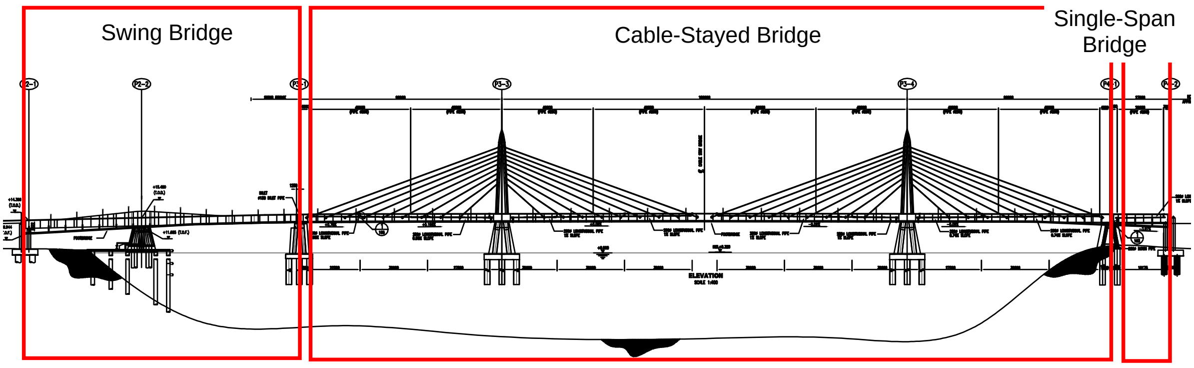

Figure 4. Main bridges of the GHMB

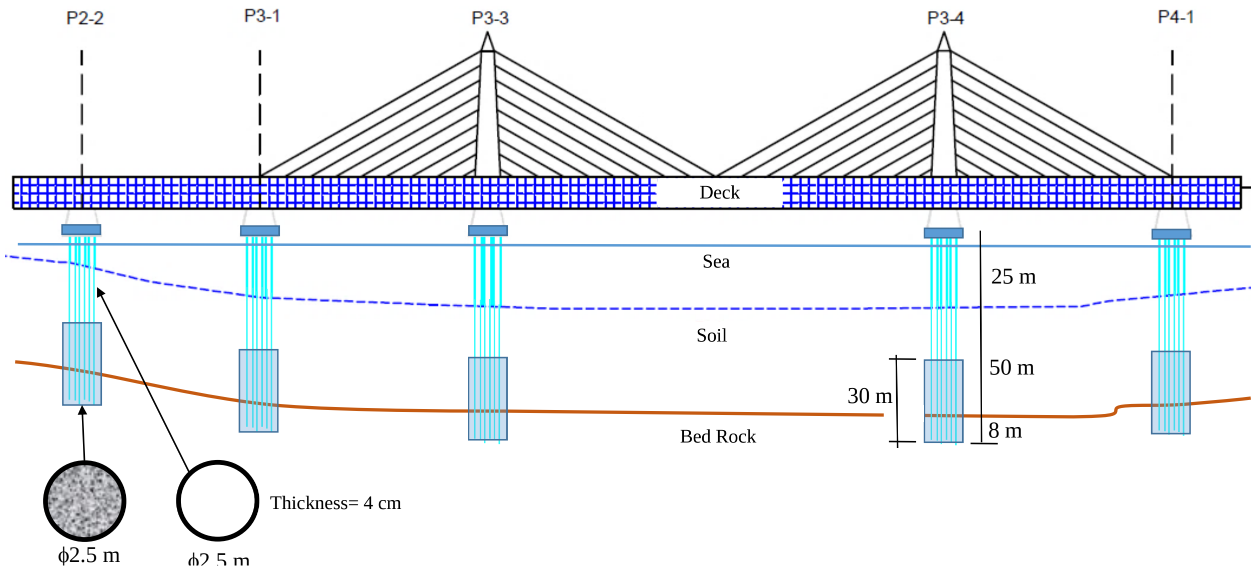

Figure 5. Schematic figure of the cable-stayed bridge

directly connected to the pylons whose height is about 54 m from the deck. Also, its sectional properties vary, but the highest section properties are 5.4 m 2.5 m and its thickness is 70 mm. Moreover, the piers are designed as different number of tubular steel piles, also the piles filled with concrete by the last 50 m (Wiecon 2011b). The design of the GHMB is conducted in line with Load Resistance Factor Design specified by AASHTO-LRFD Bridge Design Specification providing uniform safety level in terms of loads and the resistance Wiecon (2011b). The structural design properties of the Bridge is explained in Temur *et al.* (2017). The GHMB have also the permanent structural health monitoring system. This structural health monitoring system and its initial assessment is reported in Furtner *et al.* (2019)

Seismic design objective is set as "safety for running train during earthquake" and "to make resumption of transportation possible as soon as the earthquake stops". For these objectives, two levels of ground motion hazard are considered. One of them is functional evaluation of earthquake ground motion (FEE) and the other one is safety evaluation of earthquake ground motion (SEE) (Wiecon 2011b). For designing the bridge, a force-based approach is chosen in line with AASHTO-LRFD 2007 recommendations for steel bridges. According to AASHTO-LRFD , the response modification factor, R is required to be determined. The response modification factor is a combination of ductility factor and importance factor and selected as shown in Table 5. In this table, normal ductility applies to elements that are not expected to show high nonlinear behavior. On the contrary, highly ductile element are expected to experience highly nonlinear behavior. The main regions of the bridge that are expected to show nonlinear behavior are the top of piles just below the pile caps as shown later. It can be be sargued that the values shown in Table 5 are an indication of moderate damage at 2475 years return period seismic hazard. However, the design is more conservative than this level and the analyses conducted both by the designers and by this research show that bridge shows almost linear behavior under SEE event.

One of the major challenges in the design of the GHMB is the soil conditions at the piers. For Golden Horn Metro Bridge, two sets of in-situ and lab studies are conducted in 1999 and 2006. These borings are applied up to maximum 118.35 m for P3-1. According to site-specific geotechnical report, there are different types of soil types where the piles penetrated in. These soil types are presented in the geotechnical report on Golden Horn Metro Bridge (Saglamer 2009). Also, it is reported that for the pier P3-3 and P3-4, the soil models begins with the mudline, and includes very soft clay. It also can not provide lateral stiffness to support the steel piles. Soil properties are reflected to the mathematical model of the bridge as nonlinear $P-y$ springs (details are not shown herein).

Table 1. Seismic Modification factors

Ductility	Seismicity (R.P.)	
	72-years	2475-years
Normal	$R=1.0$	$R=3.5$
High	$R=1.5$	$R=5.0$

4.2. Acceleration Measurements

Acceleration data from the permanent system is used for system identification. However, due to the some discrepancies in the data, measurement on the site is taken for the study. The acceleration measurements are taken with sensors deployed on seven location on the deck (Figure 7). Distance between the accelerometers is 15 m. The measurements are taken on only the main span, where the train station is located. Acceleration records are measured in 200 Hz.

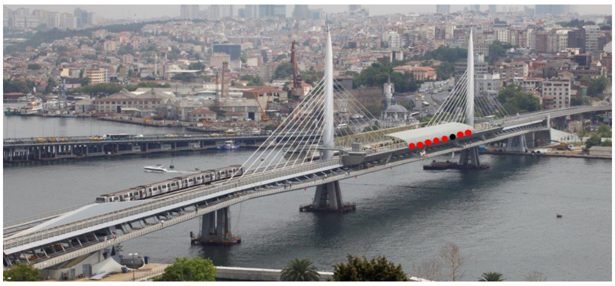

Figure 6. Acceleration measurement locations

4.3. System Identification

System identification results of the acceleration records is presented for the different system identification algorithms in Figures 9 and 10. BFD, FDD, and SSI-DATA algorithms are implemented as the system identification algorithms. Damping estimation can be calculated using the half power bandwidth method. According to this method, the following relation can be written for the first mode;

$$\xi = \frac{\omega_b - \omega_a}{2\omega_n} = \frac{0.52-0.48}{2\times 0.50} = 0.04 \tag{10}$$

Also, results of system identification methods for GHMB measurements are presented in the Tables 2 and 3 and mode shapes obtained from different system identification algorithms are reported in Figure 10.

These results show that the bridge is more stiff than the assumed mathematical model. This finding is inline with the findings of the previous system identification study conducted by Furtner *et al.* (2019). The first mode shapes are also close to the mode shape of the mathematical model with significantly large MAC numbers.

Table 2. Comparison of modal frequencies

Method	1st Mode	2nd Mode	3rd Mode	4th Mode	5th Mode
FE-Model	0.41 Hz	0.48 Hz	0.58 Hz	-	-
PP, FDD	0.50 Hz	0.68 Hz	0.99 Hz	3.09 Hz	3.64 Hz
SSI	0.52 Hz	-	-	3.12 Hz	3.66 Hz

Table 3. MAC values for the first mode

Method	MAC (%)
PP, FDD	99.75
SSI	84.00

Table 4. Damping values for the first Mode

Method	ξ (%)
PP, FDD	4.00
SSI	2.45

5. SEISMIC PERFORMANCE AND FRAGILITY ASSESSMENT OF GHMB

In this section, a sample fragility analyses is performed. For this purpose a nonlinear mathematical model is established and updated considering the results of the system identification. For one earthquake several

Figure 7. GHMB acceleration measurements

nonlinear analyses are performed to obtained a fragility curve that relates acceleration of one of the pile caps to the average damage in the piles.

5.1. Nonlinear Model

Nonlinear time-history analysis is an essential tool for performance-based design and assessment of structures. The nonlinear time history analysis is used to obtain the actual physical behavior of the structures subjected to a measured (given) earthquake. Various approaches can be used for nonlinear modelling. A general classification is lumped and distributed plasticity models. The lumped plasticity models is widely used, since these models are computationally inexpensive compared to distributed plasticity models. In the lumped plastic models, material nonlinearity buy plastic hinges or nonlinear springs and these are placed at the locations where nonlinear behavior is expected. In this paper, rigid-plastic hinges are used to assess performance of GHM Bridge.

Table 5. Target spectrum properties

Rock	
PGA (g)	0.61
T_0 (s)	0.10
T_S (s)	0.49
S_{DS} (g)	1.41
S_{D1} (g)	0.69

For the nonlinear analyses, a nonlinear model of the GHMB is developed in line with the design and the geotechnical reports Wiecon (2011b) and Wiecon (2011a). In the original nonlinear structural model, there are three types of nonlinearity: (a) geometric nonlinearity which is taken into account within the commercial program used in this study, (b) nonlinearity associated with the soil modeled as nonlinear $P-y$ springs for the boundary condition at the piles and (c) the plastic hinges that is expected to occur at the top of piles immediately below the pile capes. Nonlinear soil springs are modeled as spring with bilinear stiffness for each piles. While several models are obtained considering some or all of these nonlinearities, the results shown here includes all three types of nonlinearity. Locations of the rigid-plastic hinges and the moment-curvature relationships are shown in Figure 11. The stiffness of the model is adjusted briefly to make the fundamental mode frequency closer to the frequency obtained from the system identification study. While there are more systematic methods to update mass and stiffness matrices of the model, only stiffness of the piles of the model are adjusted by a factor since these elements are observed to have the largest impact on the frequencies. And finally it should be noted that the analyses performed in this study is not fully correct in the sense that only the bedrock the acceleration is considered. A more correct way would be identifying the soil response throughout the piles and

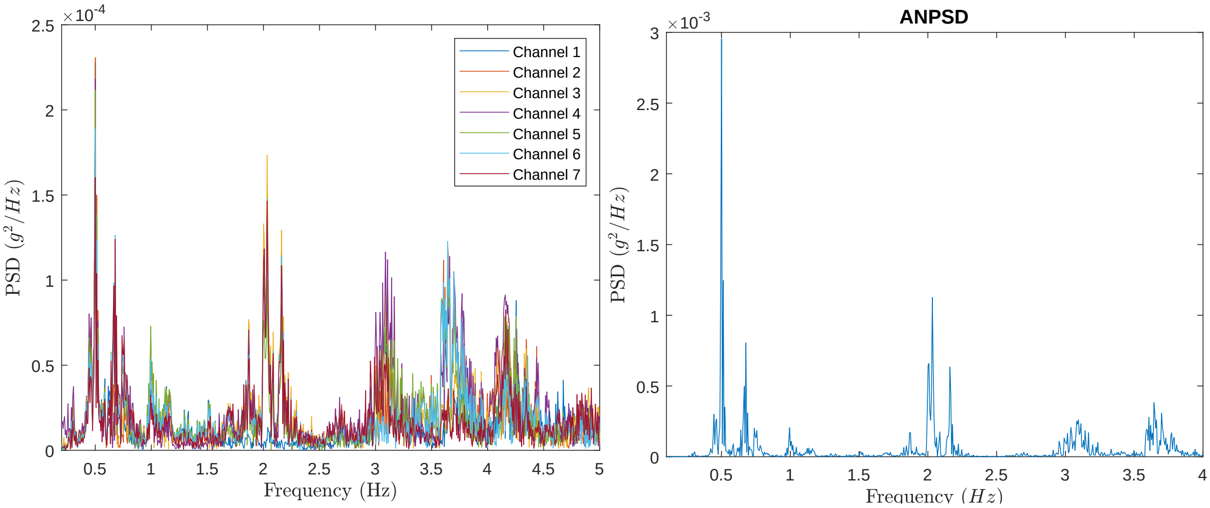

Figure 8. PSD and ANPSD of acceleration measurements

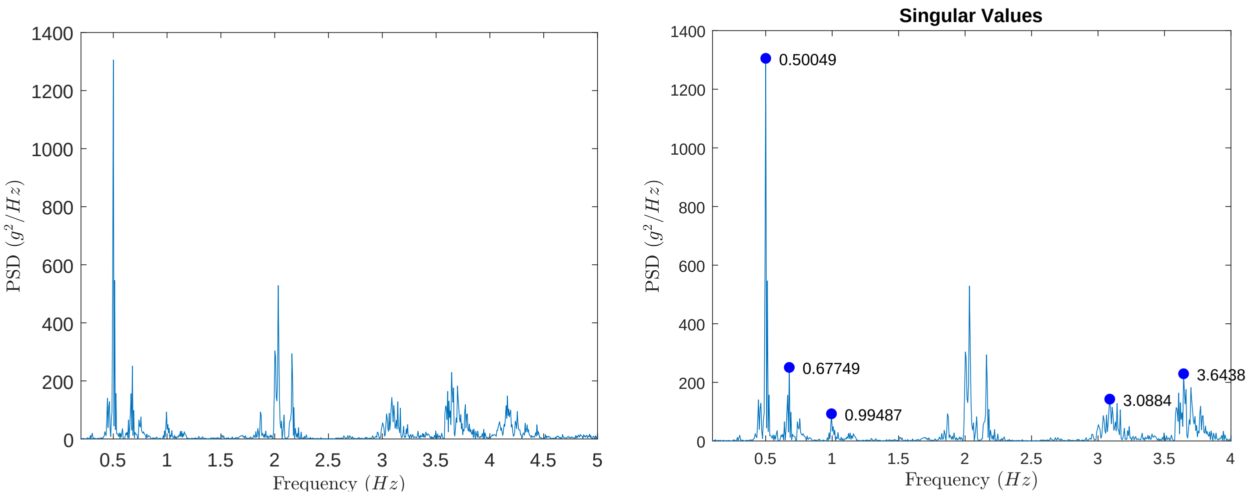

Figure 9. Trace of PSD and Singular values of acceleration records

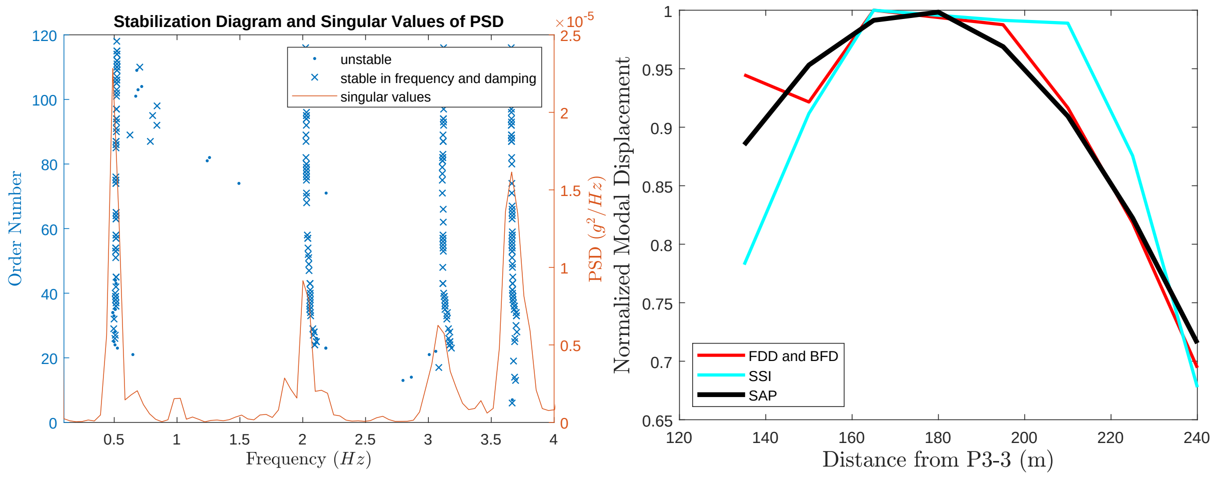

Figure 10. Stabilization Diagram of acceleration records and mode shapes

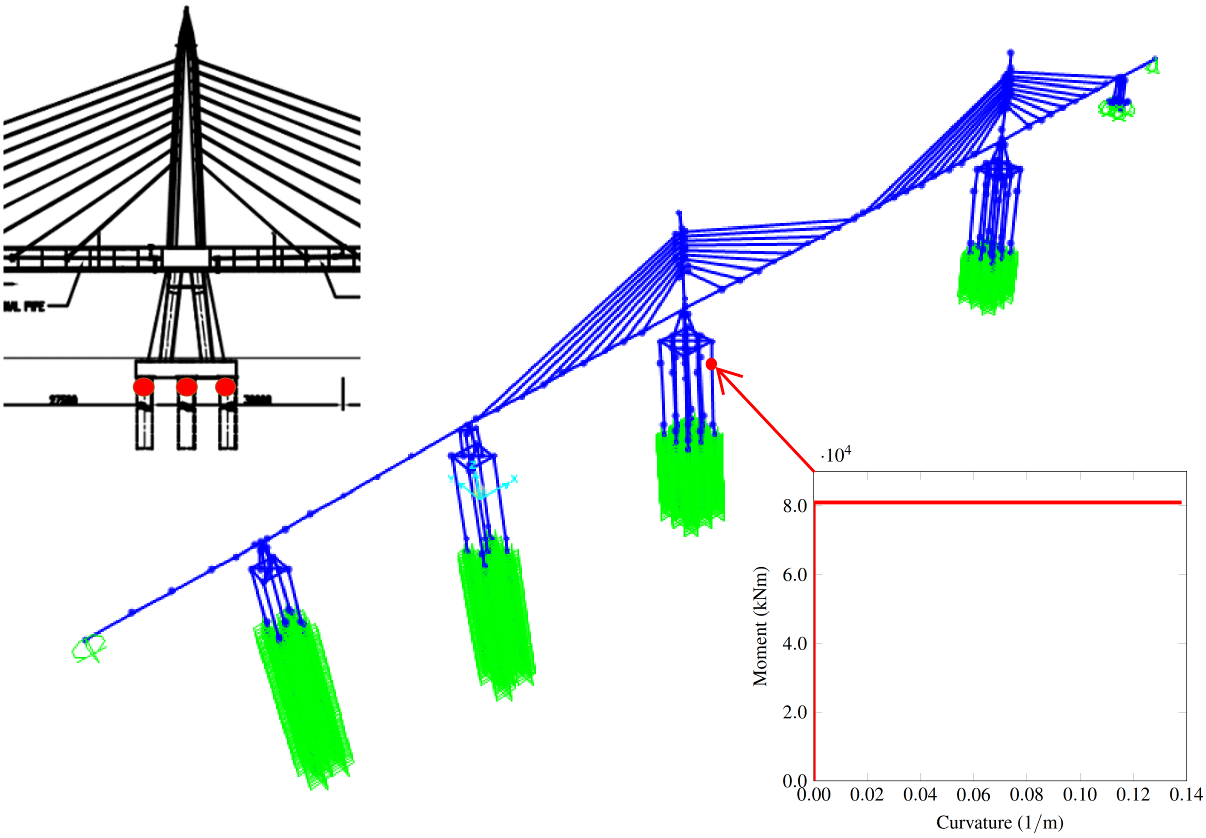

Figure 11. Location of plastic hinges and nonlinear model of GHMB

apply these response as time-histories to the supports of the $P-y$ springs. Nevertheless, the approach taken herein still provides valuable information about the structure and the use of health monitoring system is a s

5.2. Seismicity

The seismicity of the GHMB is chosen in line with the design reports (Wiecon 2011a). The report uses two types of response spectrum. The first type is at the location of bedrok, where the piles The earthquake records are matched to the rock response spectrum. The rock response spectrum are selected according to the sub-structure design report. For this purpose, the properties of the chosen response spectrum corresponding to the rock level are indicated in the table 5. Moreover, spectrum of earthquake time histories and the target spectrum is shown in Figure 12. As an earthquake time history record, 1999 Hector Mine Earthquake 1999 record RSN1762 is chosen to be used for nonlinear time history analysis. Both of two different directions of the earthquake records are matched to the target spectrum, and the nonlinear time history analyses are performed applying two direction data simultaneously.

5.3. Performance Assessment and Fragility Study

The performance of the GHMB is assessed using nonlinear time history analysis. The nonlinear time history analysis are performed using the SAP2000 commercial software (Computers and Structures, Inc. 2009). To obtain the different earthquake levels earthquake time history records are scaled by variables ranging from 1.8 to 2.4 and maximum accelerations of the top of pile P3-4 are obtained for each earthquake levels. As a performance indicator, the average plastic rotation of different plastic hinges is obtained. Finally, the results are shown in the Figure 13. As can been seen from these results, even for very large earthquakes, damage in the the structure is quite limited.

The fragility curve show a rough idea about how a seismicity can be used after a major earthquake to rapidly assess the structural condition of the bridge. However, it is clear that a more extensive study on a more through model and with many earthquake data along with the statistical information has to be performed to have a

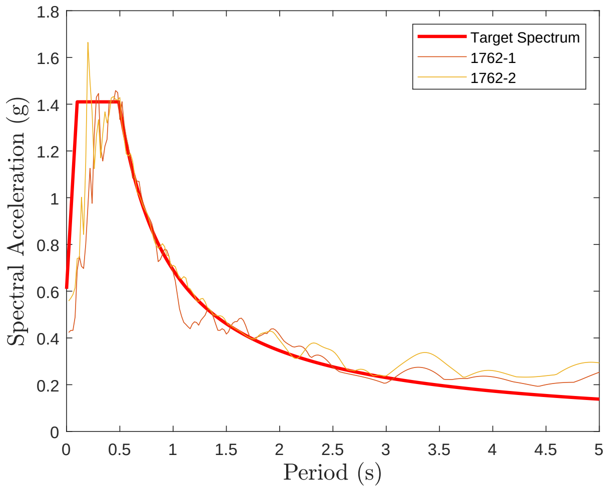

Figure 12. Target spectrum matching

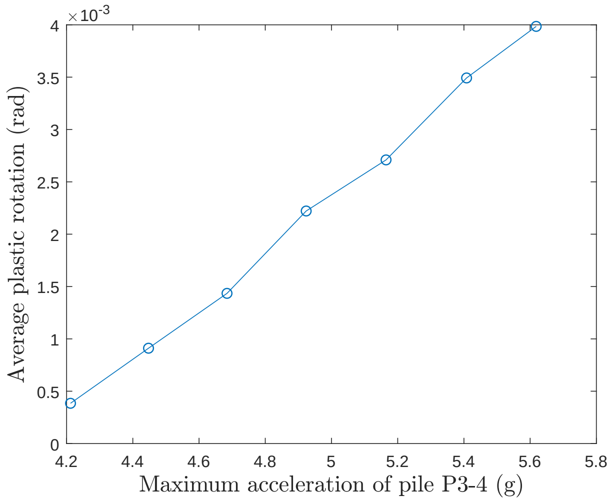

Figure 13. Sample fragility curve

realistic fragility curve; the curve presented herein is for the purpose of demonstration how fragility curves can be used.

6. DISCUSSION

This study has various findings that is considered to be useful for future studies. First of all, system identification methods are considered to be effective in identification of modal frequencies, mode shapes and damping, even for a complicated structure such as GHMB. Structural properties can be monitored at certain intervals and if there is a major or sudden change in one of the structural properties, further investigation can be initiated to avoid problematic situations. On the other hand, it is considered that it is quite challenging to identify possible locations of damage for full-scale civil structures. This is due to the fact that actual structures are quite complicated compared to the test structures, and there are significant uncertainties regarding the structural an soil properties, which are subject to significant changes throughout the lifetime of the structures. Second, it is also discussed that obtaining only modal frequencies and mode shapes using system identification after a major earthquake may not be very helpful for seismic assessment. In general, the type of changes in the structural properties of a structure after a major earthquake can be guessed to certain extend. For example, analyses on the GHMB shows that it would be normal to expect reduction in the stiffness and elongation in the period, but significant damage is not expected after a major earthquake. Another significant observation is about the health monitoring system. It is considered the health monitoring system should be establied considering the structural design and nonlinear behavior of the structure. For the case of GHMB, it would have been useful if strain at the plastic hinge locations are measured. A final observation is regarding to the complexity associated with the soil conditions. It is observed that when the soil-structure effects are significant, it is difficult to perform time-history analyses on the structure using the acceleration data collected. In the example of GHMB, the acceleration data is collected at the pile cap. However, the structure cannot be modeled placing supports at this location since influence of the piles are significant, and they also have to be modelled. In this case, the motion information throughout the piles has to be measured and fed as n input to the model. This is, on the other hand, impossible with the current health monitoring system since there are no measurements taken below the water. It would be extremely useful if such data was available; therefore it is very important that the health monitoring system should be designed considering the structural properties, design and specific issues such as soil-structure-interaction affects.

7. CONCLUSIONS

In this study, seismic health monitoring system of the GHMB is investigated. For this purpose, in-situ measurementare recorded and used for structural system identification. Nonlinear mathematical model of the GHMB is developed. System identification methods used in this study is also validated against an experimental study, where system identification is performed on a simple vibrating steel beam. The methods used in this study are, BFD, FDD and SSI. The nonlinear model of the structure is used for establishing a simple fragility curve, which relates the acceleration measures on one of the pile caps to the average plastic rotation on the top of piles. The performance of the GHMB is significantly high as damage levels are very low even for the 2475 years return period earthquake. As a result this study, it is shown that system identification algorithms developed for the GHMB are efficient and verifed against the experimental study. It is also observed that it is essential that health monitoring system should be designed and established considering the structural design and nonlinear behavior of the structure, so that data collected from the system can be utilized more efficiently.

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