EXPERIMENTAL INVESTIGATION ON DYNAMIC CHARACTERISTICS OF RC BRIDGE MODEL UNDER DIFFERENT VIBRATION LEVELS

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ABSTRACT

The present work aims to investigate the dynamic characteristics of RC pier model that derived from free vibration tests in the elastic and inelastic stages. In order to determine the changes of dynamic characteristics such as the natural frequency and the damping ratio under a series of different vibration levels, several vibration tests with a wide range of amplitudes were carried out under excitation with relatively small amplitude levels. First of all, the natural frequency and the damping ratio were obtained from free vibration test in elastic stage. Thereafter the pier model was subjected to progressive damage by conducting a shaking table test to study the influence of different damage scenario on its dynamic characteristics. At the end, to derive the natural frequency and the damping ratio of the damaged pier model, free vibration tests with different amplitude excitation were carried out again. Generally, the results of this study indicate that overall natural frequency and damping ratio are very sensitive to the vibration amplitude level. The changes in the natural frequency and the damping ratio show similar tendency in all stages. Particularly, whenever the acceleration amplitude increases, the damping ratio increases while the natural frequency decreases remarkably.

Keywords: Dynamic characteristics, Amplitude dependency, RC Pier model, Shaking table test.

1. INTRODUCTION

Overall, dynamic response of a bridge structure depends mainly on the strength, stiffness and damping properties of bridge piers as a primary source of energy dissipation. On the other hand, reliable estimates of the natural frequency and the damping ratio are essential to the prediction of dynamic response of structures which can be obtained from vibration tests such as ambient vibration, free vibration and seismic vibration. Therefore, there is a growing interest in measuring

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the structural modal parameters (natural frequencies, mode shapes and damping ratio) from dynamic measurements of structures. For instance, Peeters and Ventura (2003) presented the results of a comparative study of various techniques for evaluating bridge dynamic properties from experimental data with different types of excitations. In addition, Fukuwa et al. (1996) found that the dynamic properties strongly depend on the response amplitude when the non-structural members exist based on a full scale prefabricated steel building. However, the development of comprehensive requirements are required for the assessment of dynamic characteristics of bridges by means of free vibration test in elastic and inelastic stages to investigate non-linear effects on the dynamic characteristics. In particular, the presence of damage alters the energy dissipation mechanism in a structure; the direct consequence is that the damaged structure presents higher modal damping ratio. This change is particularly significant in the case of RC structures, where damage is usually associated with the formation of cracks (Egba Ernest 2012).

The purpose of this study is to present the results of various vibration levels for evaluating dynamic characteristics in elastic and inelastic stages of RC pier model from experimental data. To reach the inelastic stage, the pier model was excited using JMA Kushiro record of 1994 Kushiro earthquake with peak ground acceleration (PGA) of 0.7g. The seismic excitation intensity of Kushiro record was gradually increased by 10%. Herein, the excitations were repeated to clarify dynamic characteristics of RC pier model that is subjected to moderate and severe earthquake motions. During the application of seismic excitation, a free vibration test was performed after each excitation to investigate the changes of dynamic characteristics during damage progress.

2. OUTLINE OF VIBRATION TESTS

2.1. Experimental model configuration and properties

In this study, the dimension of the specimen is limited, due to the capacity of the shaking table to maintain the proper acceleration in the whole test procedure. As shown in Figure 1, the experimental model is a cantilever pier which consists of single column, cap and footing. The pier was designed based on JSCE Standard Specifications for Concrete Structures (Concrete Committee. JSCE 2007). Moreover, a concrete block was used to simulate a bridge superstructure with mass of 1.8kN. As illustrated in Figure 1, the pier cross-section is rectangular dimensions of 100mm x 150mm. The clear column height is 1500mm. The cap cross-section is with dimensions of 300mm x 350mm and a height of 100mm. In addition, the footing and the concrete block cross-sections are a square of a one side length of 600mm and a height of 200mm. The clear concrete cover is taken 15mm. During the experimental work in age of 127 days of the specimen, the compressive strength of concrete is 32.5N/mm^2, the tensile strength of concrete is 2.68N/mm^2, and the young modulus of concrete is 31.9kN/mm^2. Four-10mm diameter longitudinal reinforcing bars were provided, corresponding to a reinforcement ratio of 2.08%. 6mm diameter hoops were spaced at 100mm throughout 300mm from the pier base and its cap, and spaced at 200mm of the rest of the pier height. The yield strength of used reinforcing bars is 401N/mm^2 and the young modulus of steel is

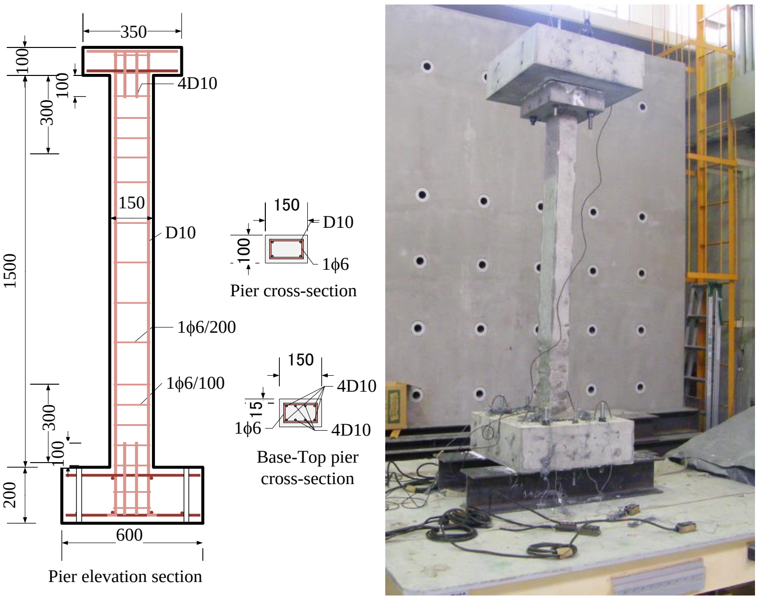
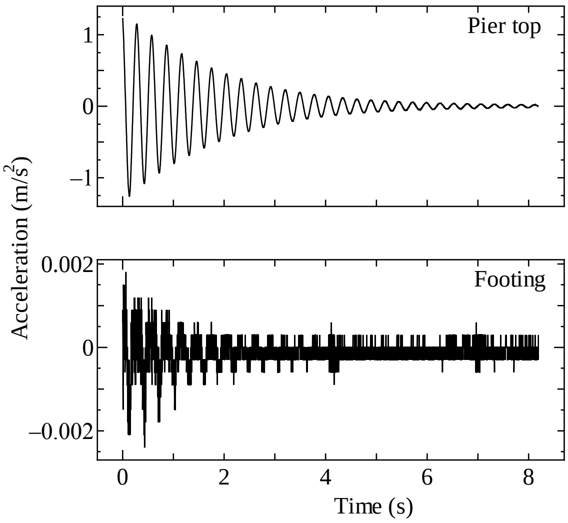


Figure 1: RC pier model

210kN/mm^2. Furthermore, strain gauges were installed on the reinforcing bars within 20cm at the pier base. On the other hand, Figure 1 shows the fixed support conditions of the longitudinal, transverse and rotational movement of the specimen. Pier footing and two steel I-girders are fixed to the shaking table by bolts. In addition, the concrete block was fixed to the pier cap through bolts.

2.2. Free vibration

The free vibration was excited by hummer impact at the top of the RC pier model with concrete block. The acceleration responses were recorded by an accelerometer in the longitudinal axis at the top of the concrete block. 8192 data points per channel were measured, resulting in a measurement period time of 8.192 s for each set-up. Furthermore, in order to determine the fixation degree of the specimen base, the acceleration responses at the top of the footing were derived in the vertical direction (Saito et al. 2013). Herein, Figure 2 shows an example of simultaneous measurement of free vibration response waveforms at the top of the pier and footing. As expected, a slight vertical vibration was observed at the footing-steel I-girder connection. This simulates the effect of soil structure interaction and energy dissipation. The identification of natural frequency from pier dynamic responses due to free vibration was obtained using Fast Fourier Transform (FFT) (Committee of Structural Engineering. JSCE 2000). One example of the acceleration response records of free vibration is shown in Figure 2 and its Fourier spectrum is shown in Figure 3. From Fourier spectrum graph, just one-dominant frequency was observed. The traditional procedure of analyzing the free decays record is used by applying band-pass filters to isolate the contribution of a single mode. The band-pass filter was employed to identify the damping ratio from the filtered free decay function by fitting exponential functions to the relative maxima of these free decays (Magalhães et al. 2012). The relationship of logarithmic acceleration versus number of cycles for the peak of each cycle in the waveform is shown in Figure 3.



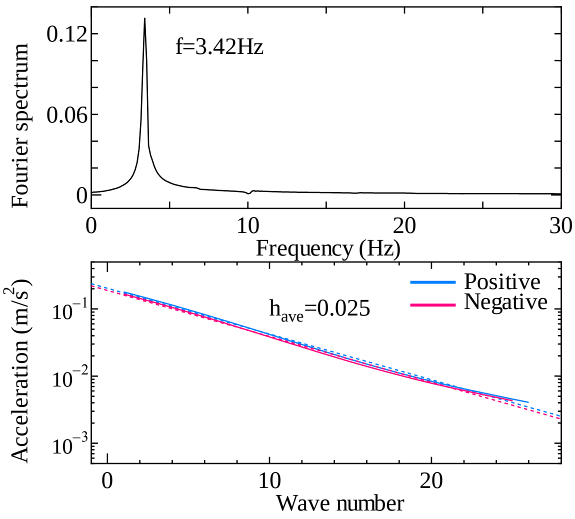


Figure 2: Acceleration time-history response

Figure 3: Fourier spectrum and acceleration vs wave number

2.3. Shaking table excitations

To represent moderate and severe earthquake motions, the specimen was excited in the longitudinal axis of the specimen by a scaled ground motion of Kushiro earthquake using JMA Kushiro record of 1994 Kushiro earthquake with peak ground acceleration (PGA) of 0.7g as shown in Figure 4. The pier model was subjected to progressive damage to study the influence of different damage scenario on its dynamic characteristics, and to investigate the variations of the natural frequency and the damping ratio of the pier model after the damage has occurred at the plastic hinge zone at the pier base. The shaking table excitation was conducted with a wide range of seismic excitation intensity of 15, 20, 25, 30%, and then the intensity was increased by 10% until reaching 160% of the peak intensity (0.7g). After each seismic excitation a free vibration test was conducted to evaluate the natural frequency and the damping ratio.

3. RESPONSE OF SHAKING TABLE TEST

Figures 5(a) and 5(b) show the Fourier spectrum of acceleration time-history response during shaking table tests under Kushiro input motion scaled to 15% and 160% respectively. The lower parts of figure 5(a) and 5(b) show Fourier spectrum of the specimen response and the upper parts show Fourier spectrum of the shaking table response in addition to Fourier spectrum response of input Kushiro ground motion. It is observed that the spectrum response of the shaking table and the input Kushiro ground motion are not identically matching each other. That is due to the effect of the specimen weight on the shaking table response which is not large enough compared to the specimen size. Moreover, it was observed a gradual decreasing of dominant frequency due to the deterioration of the specimen stiffness caused by damage sustained. Thereafter, by performing free vibration excitation after each seismic excitation, the relationships of the frequency versus the seismic excitation intensity was presented taking into account the natural frequency of free vibration test and the dominant frequency of seismic excitation as shown in Figure 6(a). This figure illustrates the decreasing of the frequency gradually with the progressing of the seismic excitation intensity. In addition, it can be observed that the dominant frequency derived from seismic excitation is not equal to the natural frequency that derived from free vibration because of the effects of the input excitation frequency on the output response. The presence of cracks in the pier model causes changes in the frequency, and then, the relationship between frequency changes and seismic intensities of Kushiro record shows that the natural frequency decreased by about 45% after the experiment. It can be said that the decreasing of the natural frequency is due to the deterioration of the specimen stiffness caused by damage sustained (G. Zafra et al. 2013). On the other hand, the damping ratio was derived from the free vibration test after each seismic excitation. The changes of the damping ratio according to seismic excitation intensities are shown in Figure 6(b). Although the damage on a structure would cause increase in damping ratio (Egba Ernest 2012), Figure 6 shows a large scattering with respect to damage progress. This is due to the dependency of the damping ratio on the response amplitude which is different according to the hummer impact for each excitation.

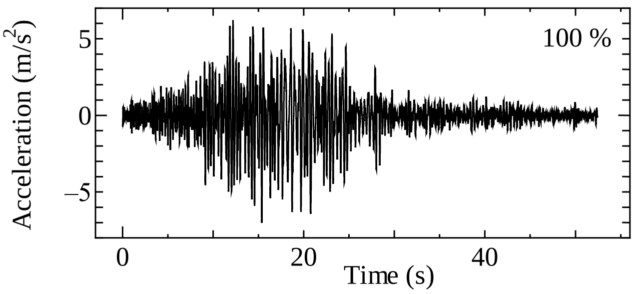
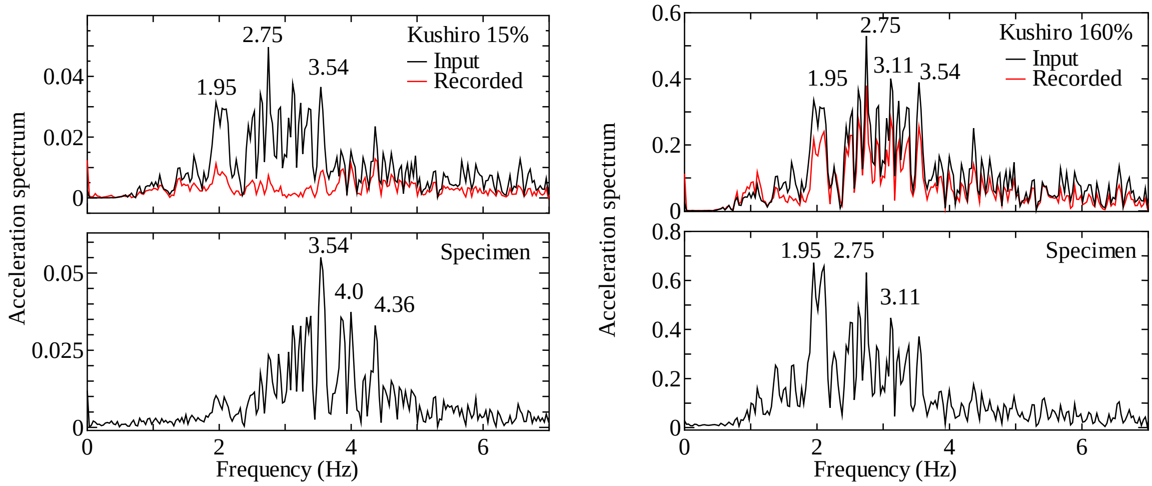
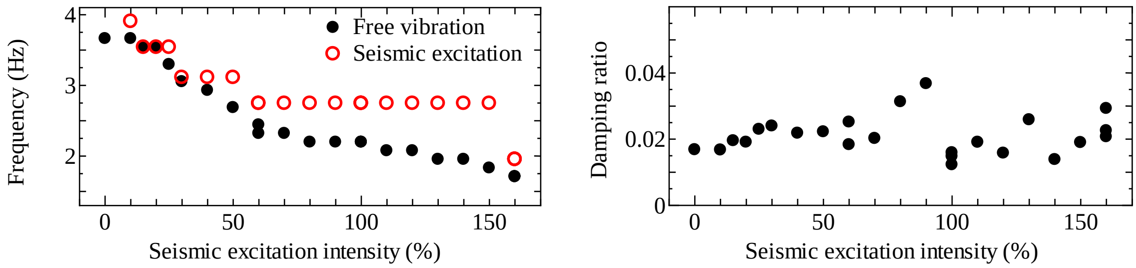


Figure 4: Time-history acceleration of Kushiro record



(a) **(b)**

Figure 5: Fourier spectrum of seismic excitation intensity



(a) **(b)**

Figure 6: Seismic excitation intensity of Kushiro earthquake versus dynamic characteristics

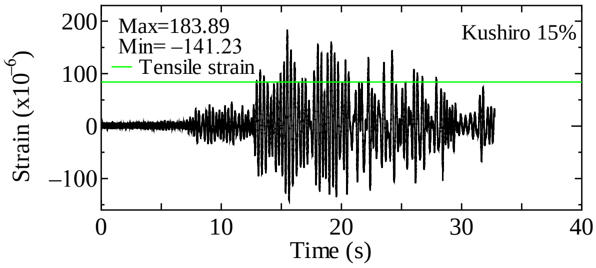
3.1. Strain time-history response

Cracks size increased by increasing the excitation gradually until the cracks width exceeded 2.0mm around the pier perimeter. Hairline vertical cracks were also observed at the plastic hinge zone within 260mm from the pier base, but the concrete cover didn't spall off. Figure 7 shows the cracks at the plastic hinge zone at the base of the pier. Strain time-history response of longitudinal reinforcement was evaluated at the plastic hinge zone near the base of the pier model. In the strain time-history response, yield strain has been defined as the strain corresponding to yield strength divided by elastic modulus of steel and tensile strain corresponding to concrete tensile strength divided by elastic modulus of concrete. Figure 8 shows one example of the strain time-history response of the pier model subjected to 15, 100, 150 and 160% of original seismic excitation. As observed from Figure 8(a), the strain response is reached the tensile strain from the first 15% of seismic excitation. The strain started yielding during 100% of seismic excitation as it can be seen in Figure 8(b), and then strain response showed a sharp strain beyond yielding in all reinforcing bars when increasing excitation intensity as in Figure 8(c). Moreover, it can be observed in Figure 8(d) that the compression strains were relatively large, thus buckling may have occurred in longitudinal reinforcements. Since the specimen will be repaired as a future work, it was not possible to investigate the damage of reinforcing bars by removing the concrete cover.

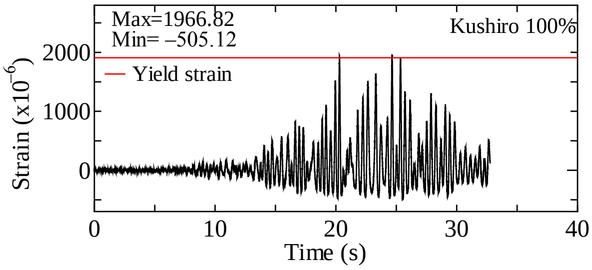
4. RESPONSE OF FREE VIBRATION TEST

In order to determine the changes of natural frequency and damping ratio under a series of different vibration levels before and after the damage of the specimen, free vibration tests of RC pier model were conducted before applying shaking table test (in elastic stage of the pier model), and then after the damage occurred in the plastic hinge zone of the specimen due shaking table test. Figure 9 shows the relationship between the maximum acceleration amplitude and the natural frequency and damping ratios, respectively, of each identified vibration level of the uncracked and cracked RC pier model. The largest acceleration amplitude could be recorded from hummer impact was about $1.26m/s^2$ derived from uncracked specimen and about $0.5m/s^2$ derived from cracked specimen. It can be observed from Figure 9(a) that as the maximum acceleration amplitude increases, the natural

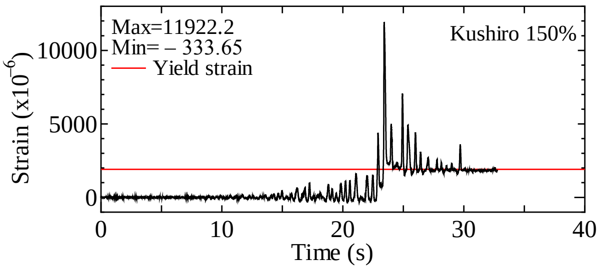
Figure 7: Cracks at plastic hinge zone



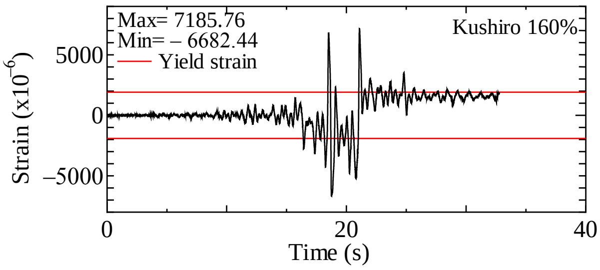
(a)



(b)

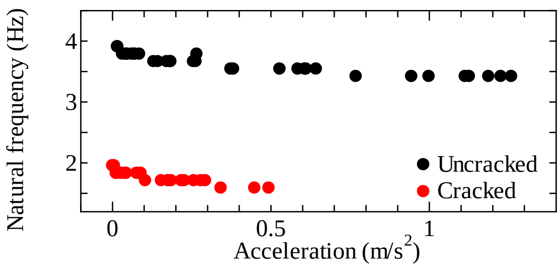


(c)

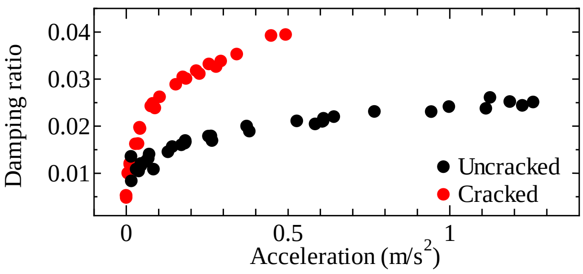


(d)

Figure 8: Strain time-history of steel at pier base subjected to different seismic intensities



(a)



(b)

Figure 9: Maximum acceleration response versus natural frequency and damping ratio

frequency tends to decrease. Figure 9(b) shows that as the maximum acceleration amplitude increases, the damping ratio increases remarkably. Moreover, a similar tendency is observed in the elastic and inelastic stage of the specimen that is the remarkable decreasing of natural frequency and increasing of damping ratio as the maximum acceleration increases. Figure 9 illustrates that in the elastic stage, the natural frequency were in range of (3.42-3.9Hz) and the damping ratio in range of (0.0083-0.026), while in the inelastic range, the natural frequency were in range of (1.59-1.95Hz) and the damping ratio in range of (0.0047-0.039).

5. CONCLUSION

A free vibration and shaking table test are conducted on RC pier model to understand the dependency of dynamic characteristics on vibration amplitude level and during the elastic and inelastic behavior of the simulated bridge model. It is concluded that when vibration intensity increases, cracks start to appear and stiffness degradation occurs. Consequently, a decrease of natural frequency and a remarkable increase of damping ratio are noticed. But, in the elastic phase of the model, the dynamic characteristics, in particular, the damping ratio have shown less dependency on vibration amplitude.

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