EFFECTS OF LONG-DURATION STRONG MOTIONS ON EARTHQUAKE RESPONSE OF HIGHWAY BRIDGES

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Abstract

A series of nonlinear dynamic response analysis of highway bridges were carried out to investigate effects of long-duration strong motions. Two nonlinear hysteretic models called HU model and N model, both taking features of post-peak behavior into account, were employed for the analyses in addition to Takeda model, which is commonly used for dynamic response analysis of RC piers. Ductility factor and residual displacement derived from the analyses in which HU and N models were used tend to be larger than those from Takeda model.

Introduction

Current Japanese design specifications require highway bridges to be checked if the bridges satisfy target seismic performances against Level 1 and Level 2 earthquake motions (Japan Road Association, 2012). Level 1 earthquake motion covers ground motion highly probable to occur during service period of bridges and its target seismic performance is set to have no damage. Level 2 earthquake motion is defined as ground motion with high intensity with less probability to occur during the service period of bridges. The target seismic performance against Level 2 earthquake motion is set to prevent fatal damage for bridges with standard importance and to limit damage for bridges with high importance.

Seismic design of highway bridges are carried out without considering pinching behavior and strength degradation because the limit state corresponding to the target seismic performance against Level 2 earthquake motions is set to be within stable hysteresis loop according to results from cyclic loading tests.

In this study, a series of nonlinear dynamic response analysis were carried out to investigate effects of long-duration strong motions on post-peak earthquake response of highway bridges. Two nonlinear hysteretic models proposed by Hoshikuma and Unjoh (2001) and Nogami et al. (2008) were employed for the analyses in addition to Takeda model (1970), which is commonly used for dynamic response analysis of RC piers.

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Input Motions

There are two types of Level 2 earthquake motion, i.e. Type I and Type II earthquake motions, in the design specifications. Type I represents ground motions from large-scale plate boundary earthquakes, while Type II from inland earthquakes and directly strike the bridges. These design earthquake motions are defined as design acceleration response spectra with damping ratio of 0.05. Time history waveforms are also given in the design specifications for seismic design using dynamic response analyses. The time history waveforms were produced by spectral fitting using strong motion records as original waveforms; their acceleration response spectra were adjusted to fit to the design spectra by means of a spectral fitting technique.

Three acceleration waveforms are given for each type of Level 2 earthquake motion and each of three ground types, i. e. Ground Type I, II, and III corresponding to hard, intermediate, and soft ground conditions, respectively. Figure 1 shows the waveforms employed for the dynamic analyses in this study. The waveforms represent strong motion at soft ground condition, Ground Type III. The amplitudes of the waveforms of Type I earthquake motions were amplified by multiplying 1.0, 1.25, 1.5, 1.75, and 2.0 in order to investigate effects of strong motion intensity. Those of Type II earthquake motions were also amplified by multiplying 1.0 and 2.0. Note that the duration of Type I earthquake motion is 120s or 240s while that of Type II is 50s.

Analytical Model of a Highway Bridge

Figure 2 shows the analytical model of a pier of a highway bridge with the superstructure, fixed bearing, and pile foundation. The height of the pier was set to 10m. The bridge was designed to be built on soft ground (Ground Type III) under the current specifications. The natural period of the model was 0.76 and damping factors 0.02 and 0.2 were given to the pier and the pile foundation, respectively.

The model proposed by Hoshikuma and Unjoh (2001), which is referred to as HU model, takes pinching behavior into account, while Nogami et al. (2008) proposed another model, which is referred to as N model, considering strength degradation. The nonlinear hysteretic models for the plastic hinge section of the pier including Takeda model, which is referred to as T model, are shown in Figure 3. Figure 4 compares hysteresis loops obtained from a cyclic loading test (Hoshikuma et al., 2013) and HU and N models. The hysteretic models show a noticeable difference although they were proposed for the same purpose, representing post-peak behavior of RC piers.

Analytical Hysteresis Response

Figure 5 shows hysteresis response of the RC pier subjected to the waveform II-1. The waveform is based on the strong motion recorded near Higashi-Kobe Ohashi Bridge during the 1995 Kobe earthquake (Mw6.9). The response loops do not display obvious difference depending on the hysteretic models even if the amplitude of the input motion was doubled. The ductility factors are 15.6, 17.5, and 15.3 and the residual displacements are 0.10m, 0.30m, and 0.11m for T, HU, and N models, respectively.

Figure 6 shows the hysteresis response of the RC pier subjected to the waveform I-1. The waveform is based on the strong motion recorded at Taiki town station, Hokkaido during the 2003 off Tokachi earthquake (Mw8.0). The response loops show remarkable difference depending on the models. The ductility factors are 11.4, 14.5, and 15.2 and the residual displacements are 0.07m, 0.14m, and 0.10m for T, HU, and N models, respectively.

Figure 7 shows the hysteresis response of the RC pier subjected to the waveform I-2. The waveform is based on the strong motion recorded at Yamazaki station, Miyagi during the 2011 off the Pacific of Tohoku earthquake (Mw9.0). It has two strong motion parts last 40 s each. The response loops show remarkable difference as well as in Figure 6. The ductility factors are 10.2, 13.8, and 26.6 and the residual displacements are 0.04m, 0.33m, and 0.27m for T, HU, and N models, respectively. The differences of the ductility factors and the residual displacements between T model and HU and N models become larger when the waveform I-2, which has longer duration, is used as the input motion.

Comparison of the Hysteresis models

Figures 8 and 9 compare the residual displacements. Figure 8 compares T and HU models while Figure 9 compares T and N models. The residual displacements tend to be larger for HU and N models, especially in the case of N model and the waveform I-2.

Figures 10 and 11 compare the ductility factors. There is little difference among the models in the case of Type II earthquake motions. On the other hand, in the case of Type I earthquake motions, the ductility factors tend to be larger for HU and N models, especially in the case of N model and the waveform I-2.

Conclusions

A series of nonlinear dynamic response analysis of highway bridges were carried out to investigate effects of long-duration strong motions. Two nonlinear hysteretic models called HU model and N model, both taking features of post-peak behavior into account, were employed for the analyses in addition to Takeda model, which is commonly used for dynamic response analysis of RC piers. Ductility factor and residual displacement derived from the analyses in which HU and N models were used tend to be larger than those from Takeda model. The differences become more remarkable when the waveform with longer duration was employed for the analyses.

References

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Hoshikuma, J. and Unjoh, S. (2001): Earthquake response characteristics of RC piers considering post-peak behavior, Proceedings of the 26th Earthquake Engineering Meeting, Japan Society of Civil Engineers, pp. 997-1000.

Japan Road Association (2012): Specifications for Highway Bridges and the Commentary, Part V: Seismic Design

Nogami, Y., Murono, Y. and Sato, T. (2008): Nonlinear hysteresis model considering strength degradation by cyclic loading for RC members, QR of RTRI, Vol. 49, No. 4, pp. 227-231.

Takeda, T., Sozen, M. A., Nielsen, N. N. (1970): Reinforced concrete response to simulated earthquakes, ASCE J. Struct. Div., Vol. 96, No. 12, pp. 2557-2573, 1970.

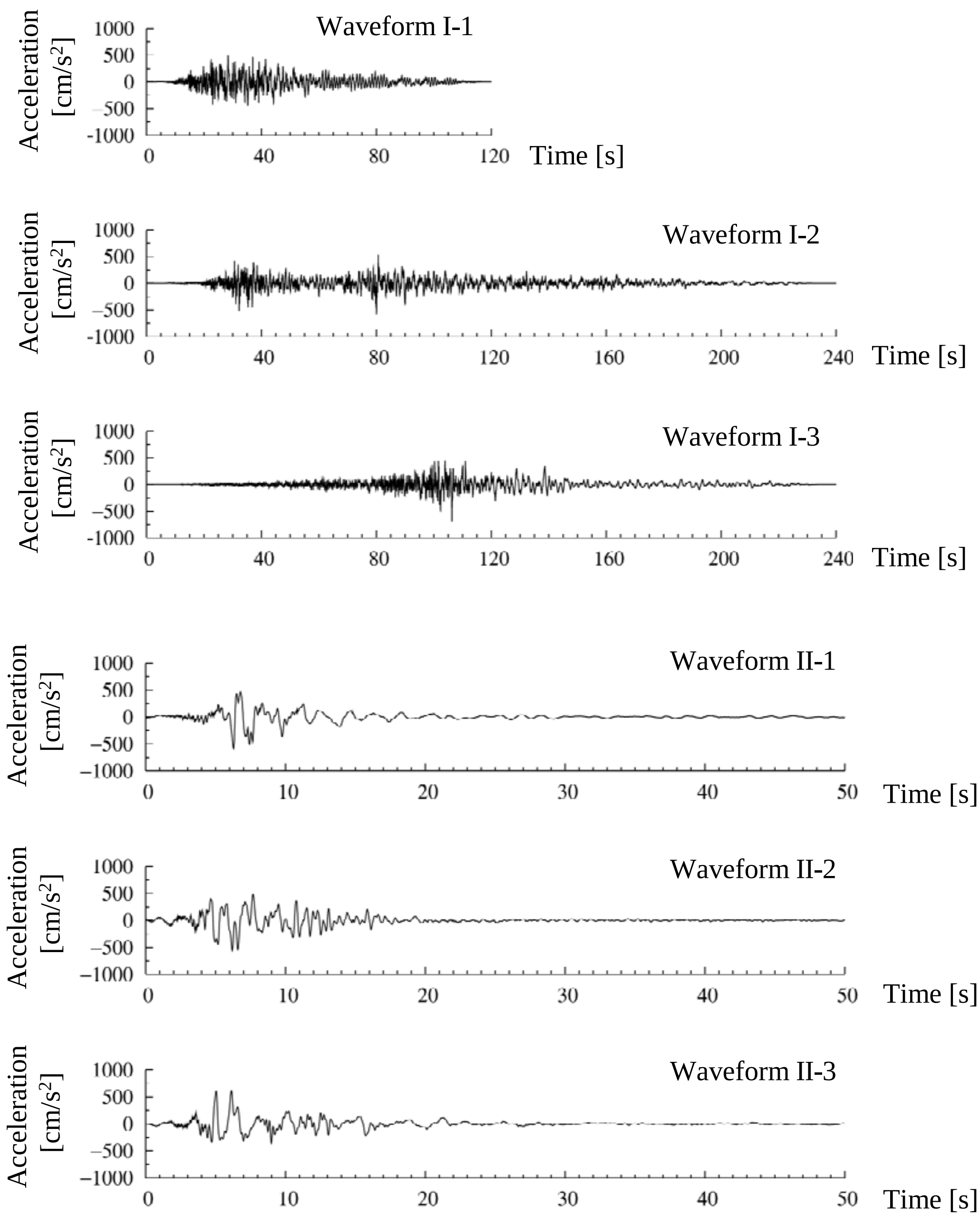


Figure 1 Waveforms used for the nonlinear dynamic analyses. Waveforms I-1, I-2 and I-3 are earthquake motions at Type III ground from large-scale plate boundary earthquakes, while waveforms II-1, II-2 and II-3 are those from inland earthquakes.

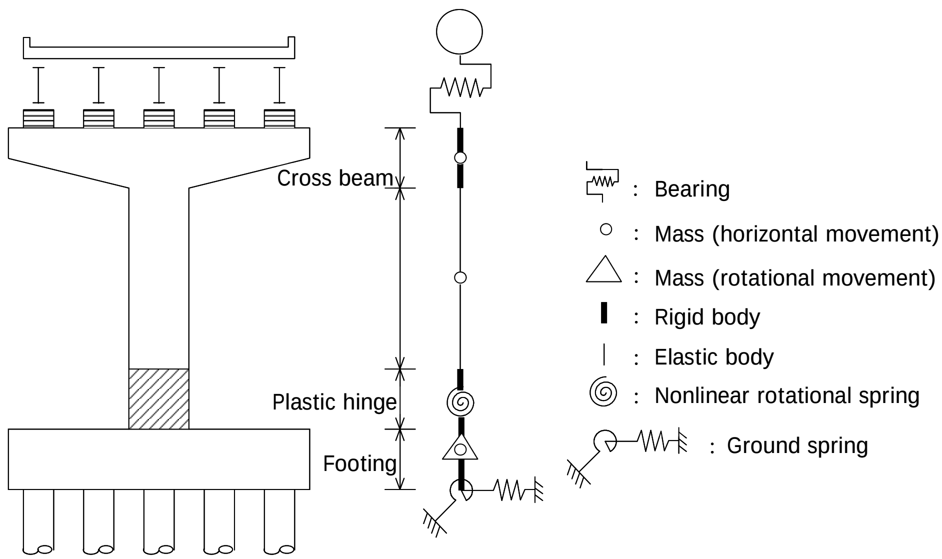
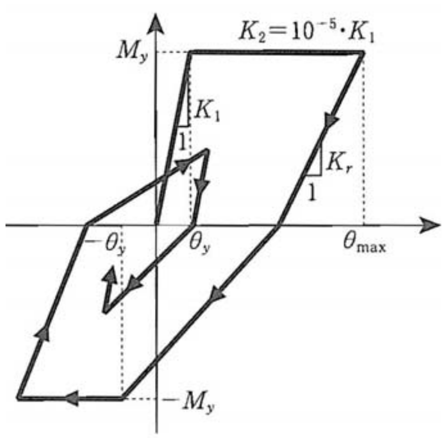
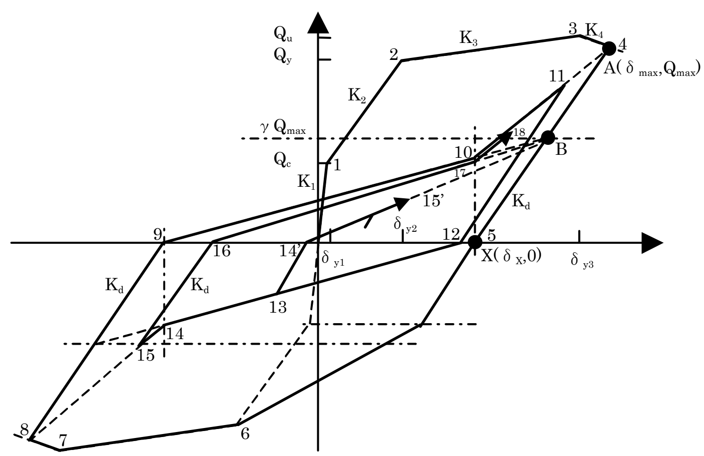


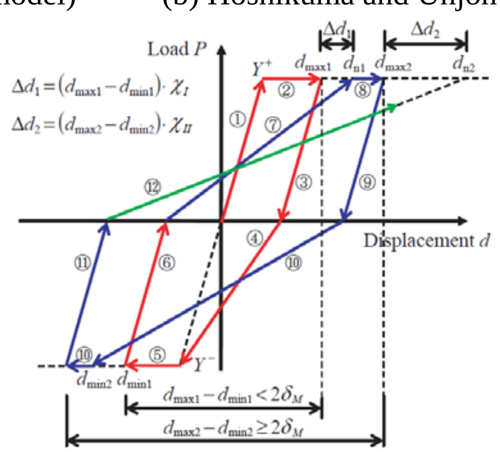
Figure 2 Analytical model of a highway bridge.



(a) Takeda model (T model)

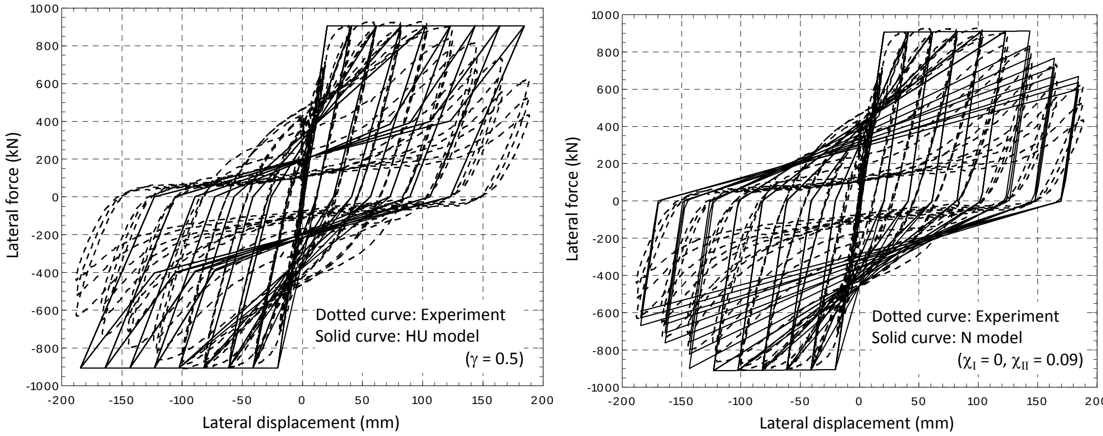


(b) Hoshikuma and Unjoh model (HU model)



(c) Nogami et al. model (N model)

Figure 3 Nonlinear hysteresis models used in this study.



(a) HU model (b) N model

Figure 4 Comparison between hysteresis loops obtained from a cyclic loading test and the hysteresis models.

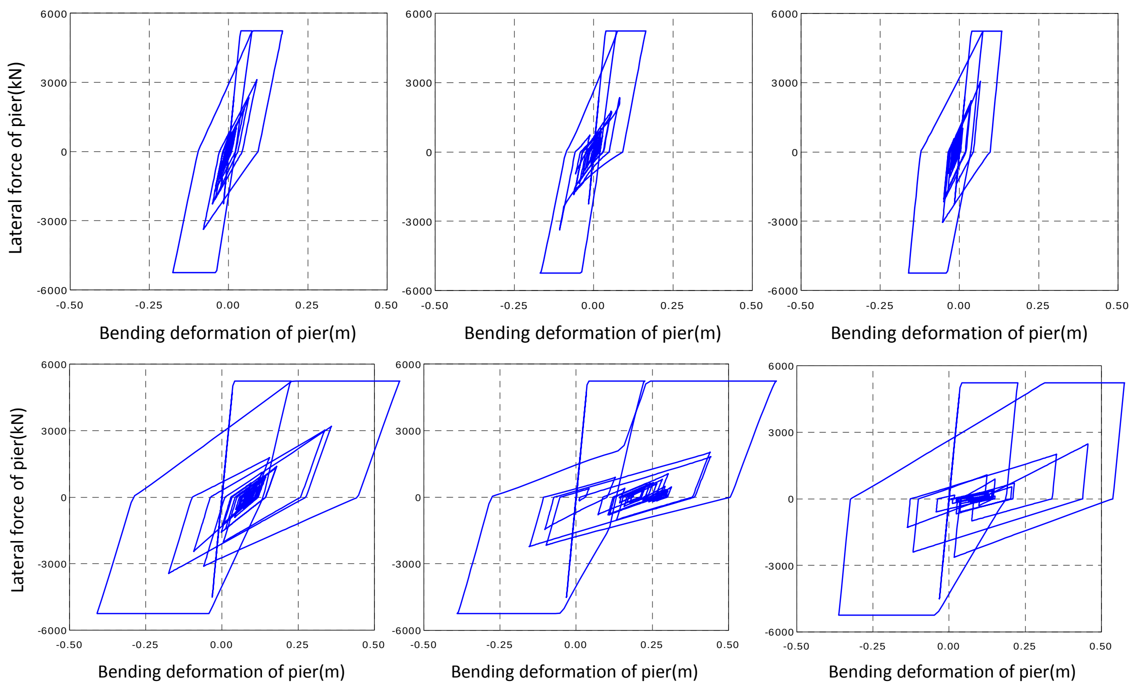


Figure 5 Hysteresis response of the RC pier subjected to the waveform II-1 (top: x1.0, bottom: x2.0).

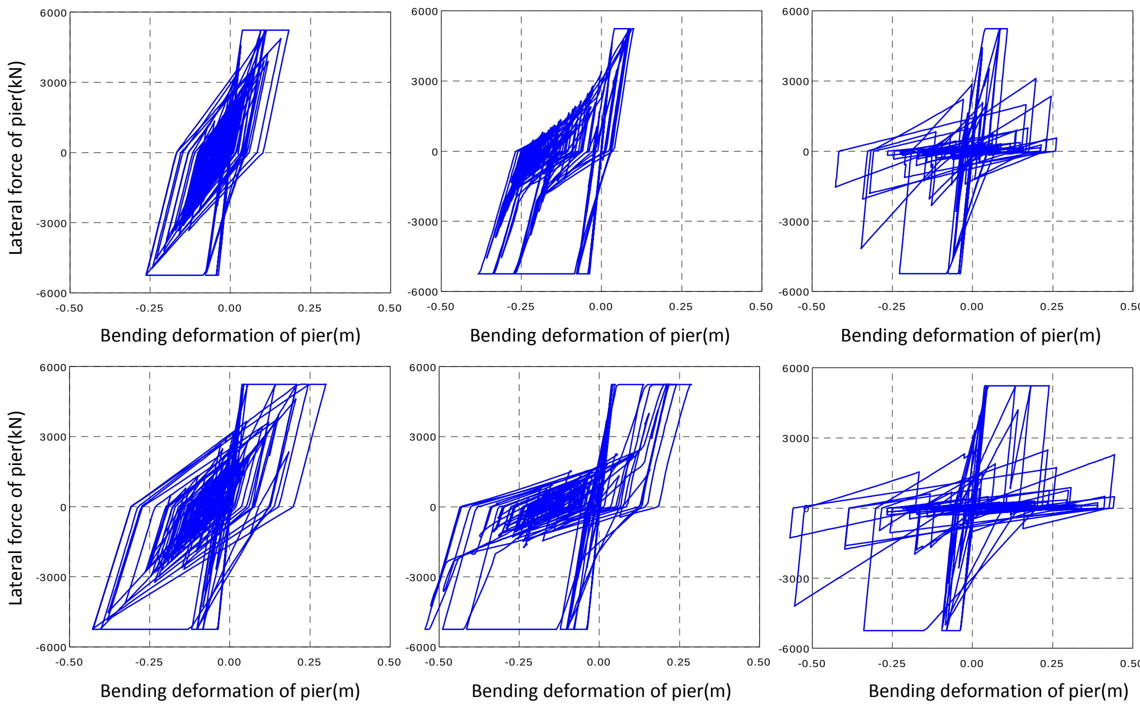


Figure 6 Hysteresis response of the RC pier subjected to the waveform I-1 (top: x1.5, bottom: x2.0)

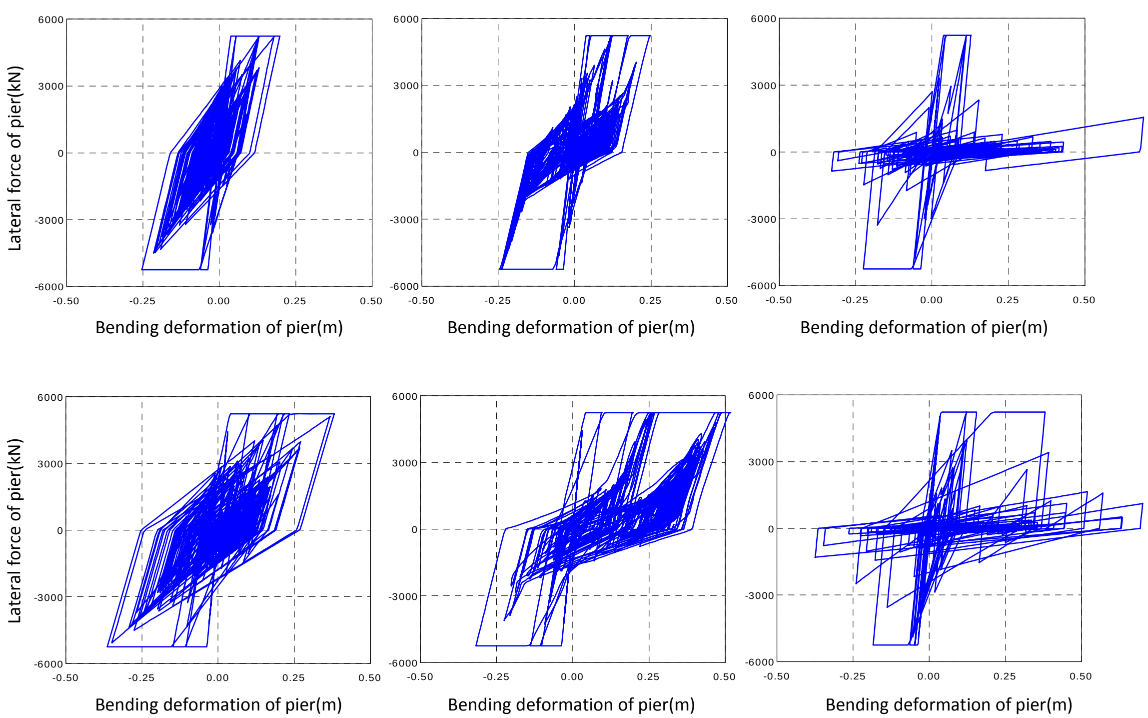


Figure 7 Hysteresis response of the RC pier subjected to the waveform I-2 (top: x1.0, bottom: x2.0).

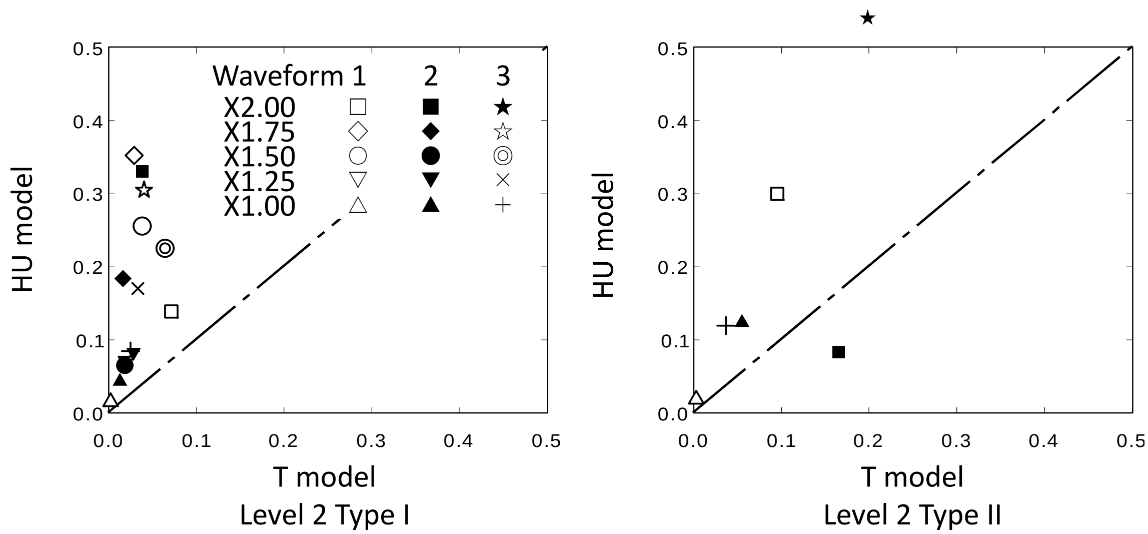


Figure 8 Comparison of the residual displacements of T and HU models.

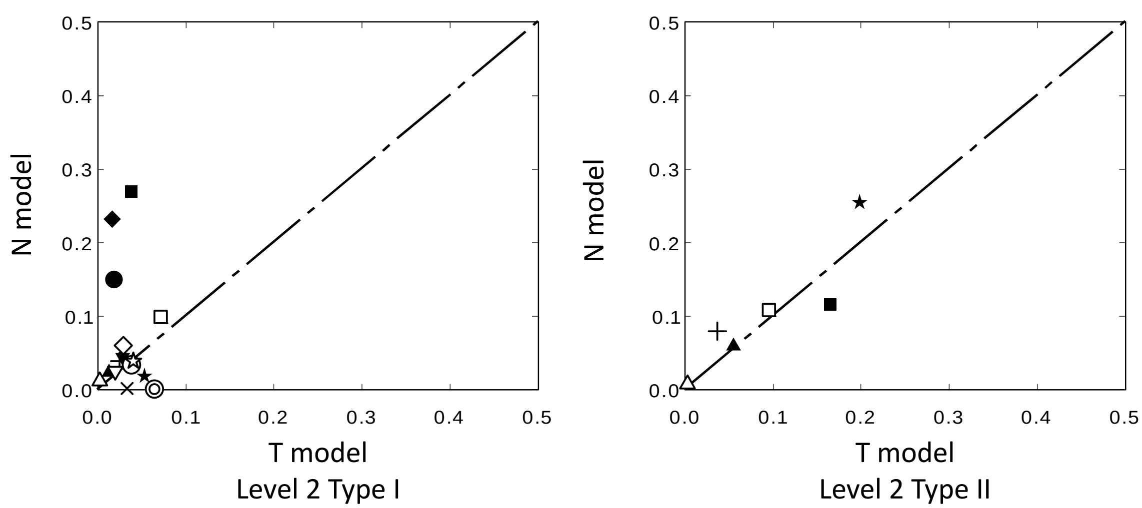


Figure 9 Comparison of the residual displacements of T and N models.

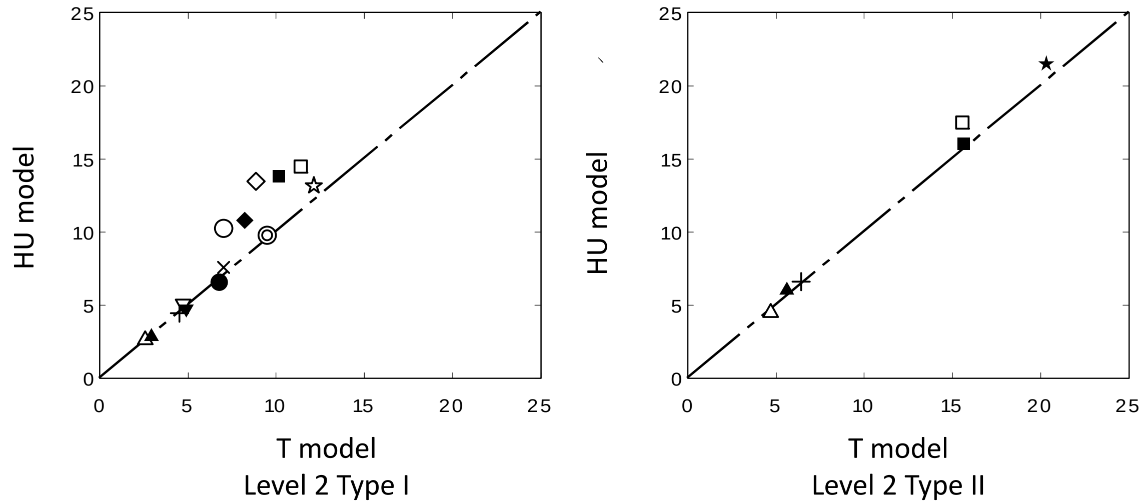


Figure 10 Comparison of the peak response ductility factors of T and HU models.

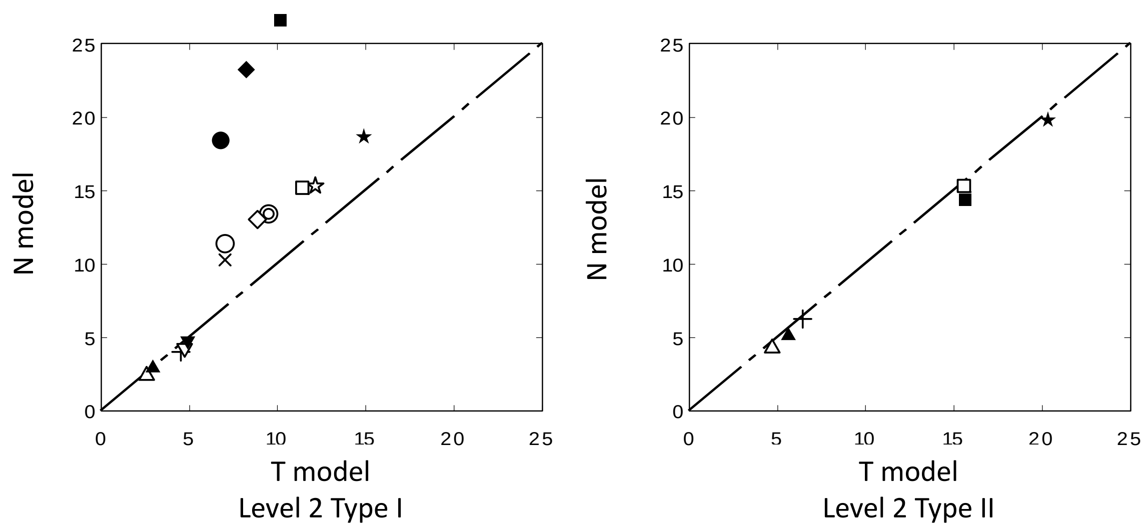


Figure 11 Comparison of the peak response ductility factors of T and N models.