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A Fundamental Study on the Effect of Dynamic Deformation Property Testing method in Seismic Design for a Bridge Pier

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Abstract

In the Japanese seismic design of railway structures, inertial force and ground displacement applied to a target structure are calculated by a time domain nonlinear dynamic ground response analysis. For determining nonlinear parameters of soils necessary for the analysis, such as G/G_0~τ and h~τ relationships, cyclic shear tests are conducted using a tri-axial compression test apparatus, a torsion shear test apparats and so on. "The stage loading testing method" is generally adopted as a loading scheme, in which soil specimen is repeatedly sheared 11 times at each strain level and deformation properties are calculated using a hysteresis loop of 10th cycle. This method was developed in order to determine the parameters mainly for small-to-medium shear strain level ($10^{-5} < \gamma < 10^{-3}$) in the late 1960s. On the other hand, large shear strain level has to be considered in the current seismic design of structures against a large-scale earthquake, such as a level 2 earthquake. Some researchers, therefore, have pointed out some problems of the conventional stage loading testing method.

The authors have proposed a new testing method, which is composed of two types of testing method: a strain-controlled 1 cycle stage shear test and strain-controlled constant strain cyclic shear test. The past study clearly showed that the dynamic ground response analysis with the G/G_0-γ and h-γ curves obtained from the proposed method could produce almost the same seismic ground displacement with that of the hybrid ground response simulation which could give the most accurate seismic ground response although relatively small response acceleration might be calculated. On the other hand, it was also confirmed that the conventional testing method might give extremely small shear stiffness for a large strain level.

This study evaluates the effect of the above mentioned discrepancy in the results of ground response analyses attributed to different testing methods for obaining nonlinear deformation properties on a seismic design of a railway bridge pier by a pseudo static nonlinear analysis.

The test results of the trial design indicated that the conservative evaluation of the inertial force in the proposed method comparing to that of the hybrid ground response simulation could not give large effect on the response of the bridge pier. In addition, it was confirmed that deep foundation might be designed with extremely over spec if the dynamic deformation properties were obtained from the conventional method.

Keywords: seismic design, railway bridge pier, ground response analysis, deformation characteristics of soils

1. Introduction

In the Japanese seismic design of railway structures, it is verified that structures show required performance against the design seismic motion by checking that design response values of structures are less than design limit values. Design response values of structures should be calculated using appropriate calculation models which can express their seismic behavior in consideration of interaction between surface ground and structures, deformation properties of members and surface ground and so on. In addition, "inertial force" and "ground displacement" should be carefully determined as actions applied to calculation models. Inertial force and ground displacement acted on structures, that is, seismic ground resoponse is significantly affected by the configuration of surface ground stratification and nonlinearity of soils composing of surface ground. 'The Japanese Seismic Design Standard for Railway Structures'[1] recommeds to conduct a time-domain nonliear dynamic ground response analysis for determing seismic ground response.. In the time-domain dynamic ground response analysis, the GHE-S model is also recommended to use for appropriately expressing nonlinear deformation properties of a wide range of strain level, such as G/G_0~γ and h~ γ relationships, where G is shear stiffness; G_0, initial shear stiffness; h, hysteresis damping; and γ, shear strain. For determining G/G_0~γ and h~ γ relationships, cyclic shear tests of soils are usually conducted using a tri-axial compression test apparatus, a torsion shear test apparats and so on. As a loading method, "Stage loading testing method" is generally adopted as a loading scheme, in which soil specimen is repeatedly sheared 11 times at each strain level and deformation properties are calculated using a hysteresis loop of 10th cycle. This method was developed in order to determine the parameters mainly for small-to-medium shear strain level ($10^{-5} < \gamma < 10^{-3}$) in the late 1960s. On the other hand, large shear strain level ($10^{1} < \gamma$) has to be considered in the current seismic design of structures against a large-scale earthquake, such as a level 2 earthquake. Some researchers, therefore, have pointed out some problems of the conventional stage loading testing method.

The authors have proposed a new soil testing method[4] to determine G/G_0-γ and h-γ relationships necessary for a time-domain nonlinear dynamic ground response analysis against a large-scale earthquake. The proposed method is composed of two types of testing method: a strain-controlled 1 cycle stage shear test and strain-controlled constant strain cyclic shear test. The past study clearly showed that the dynamic ground response analysis with the G/G_0-γ and h-γ curves obtained from the proposed method could produce almost the same seismic ground response with that of the hybrid ground response simulation (called 'HGRS' in short), which can give the most accurate seismic ground response[5), 6)], although relatively small response acceleration might be calculated. On the other hand, it was also confirmed that conventional "Stage loading testing method" might give extremely small shear stiffness for a large strain level.

This study evaluates the effect of the above mentioned difference in results of time-domain nonlinear dynamic ground response analyses on a seismic design of a railway bridge pier, attributed to difference of testing methods for nonlinear deformation properties.

2. Evaluation for seismic response of surface ground

The conventional and proposed methods were applied for the Toyoura sand with relative density of 60%[5), 6)]. The τ-γ and h-γ relationships obtained are shown in Fig. 1, solid lines in which indicate fitting results by the GHE-S model. The results clearly showed that the conventional method tended to give smaller shear stiffness ratio at large strain level and smaller hysteresis damping at wide strain length. In addition, the GHE-S model can adequately fit the deformation properties obtained from the proposed and the conventional cyclic tests except at large strain level of the conventional test which is extremely underestimated by the test.

Two cases of time domain ground response analyses were conducted for the same model ground shown in Fig. 2, in which nonlinear deformation properties obtained from the conventional (CASE 1) and the proposed (CASE 2) tests were applied to the target layer. The results of the analyses were indicated in Fig. 2 and 3, together with results of the HGRS which can give the most accurate seismic ground response. In the analyses, the level 2 earthquake (spectrum II) of the seismic design standard[1] was applied to the model as shown in Fig. 4. As a result, the following conclusions were obtained.

1) Ground displacements using nonlinear deformation properties obtained by the proposed testing method were almost the same with that in the HGRS (Fig. 2).

2) The maximum acceleration at the ground surface obtained from the ground response analysis with the proposed method tended to be underestimated as compared with that from the HGRS(Fig. 2 and 3).

It was confirmed that the proposed soil testing method was more adequate to evaluate for ground displacement with the dynamic ground response analysis against a large-scale earthquake although it could give smaller response acceleration. Large acceleration observed in the HGRS was attributed to cyclic mobility of soil, which cannot be simulated by the total stress model, such as the GHE-S model. The effect of such large discrepancy in the response acceleration for seismic design of structures is considered in 3.1.

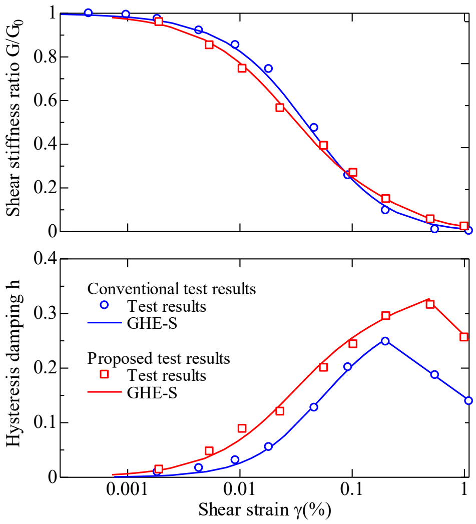


Fig. 1 Deformation properteis of Toyoura sand(Dr=60%s)

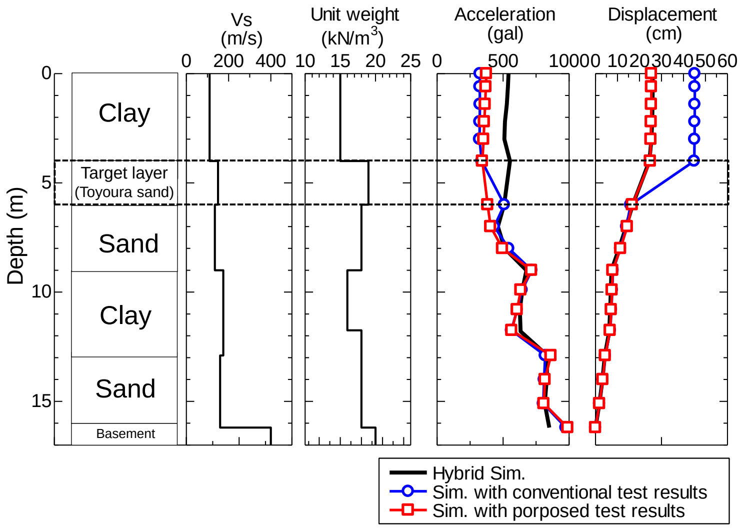


Fig. 2 Distribution of the maximum response valuesobtained from ground response analysis

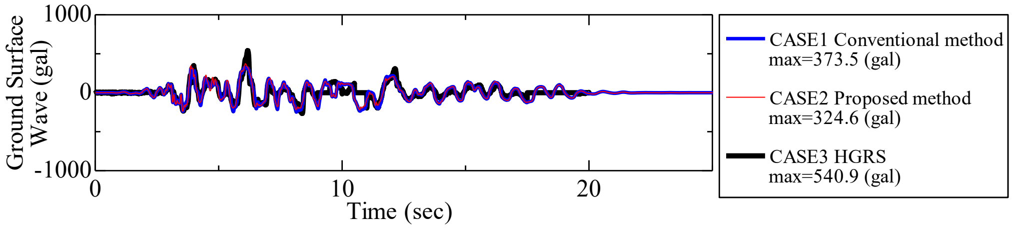
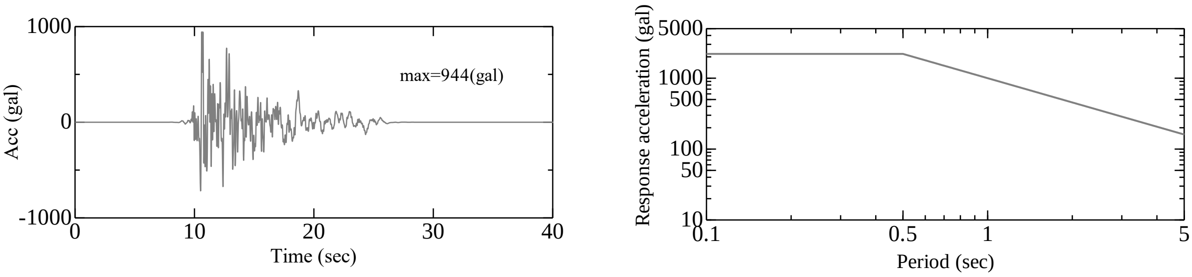


Fig. 3 Time histories of repsonse acceleration at surface obtained from the ground response analyses



(a) Time-history

(b) Elastic acceleration response spectrum

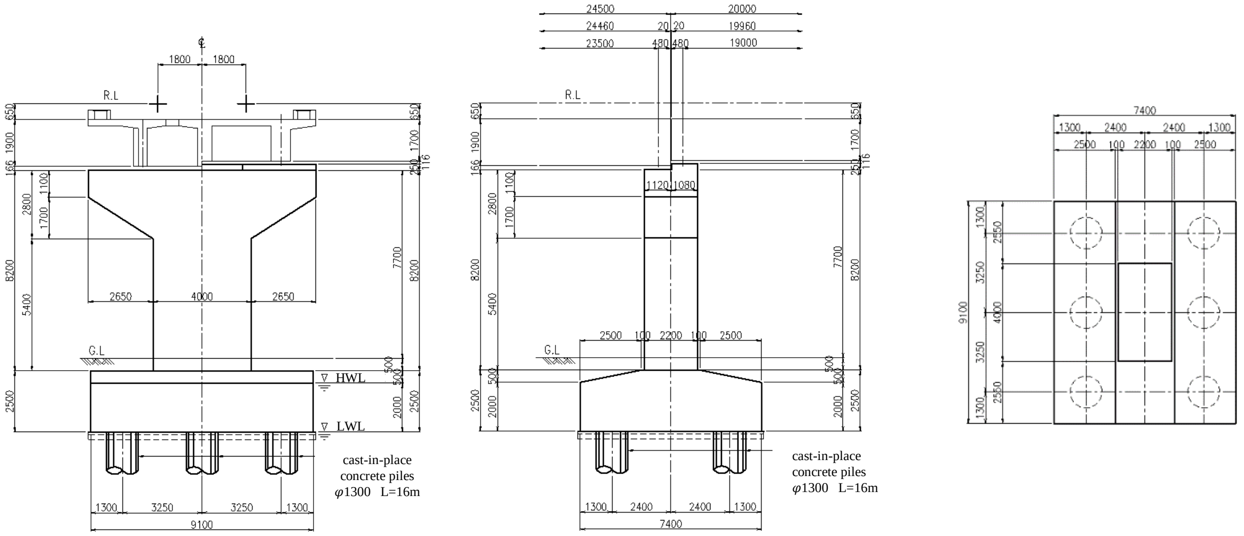
Fig. 4 The level 2 earthquake (spectrum II)[1)]

3. Evaluation for seismic response of a bridge pier

Seismic response values of a railway bridge pier were calculated based on psudo static nonlinear structural analysis, in which 3 cases were conducted using different seismic actions obtained from the different ground ground response analysis described in Chapter 3.

3.1 Modeling of the Bridge Pier and the Surface Ground

The bridge pier used in this design was a railway bridge pier supported by 2×3 cast-in-place concrete piles (pile diameter: 1.3m, pile length: 16.0m), and the height of which was 8.2m as shown in Fig. 5. The nonlinear pseudo static seismic response analysis was used to calculate the design response values based on the Japanese railway standard. The bridge pier was modeled by a mass-spring model composed of 54 beam elements and spring elements as shown in Fig. 6. Nonlinearities of the concrete members and the soil springs were modeled as the tetra-linear type and the bi-linear type respectively (Figs. 7 and 8).



(a) Front view (b) Side view (c) Plan view

Fig. 5 Schematic view of the bridge pier used in the trial design.

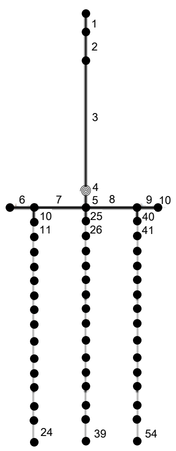


Fig. 6 Mass-spring model used in the trial design

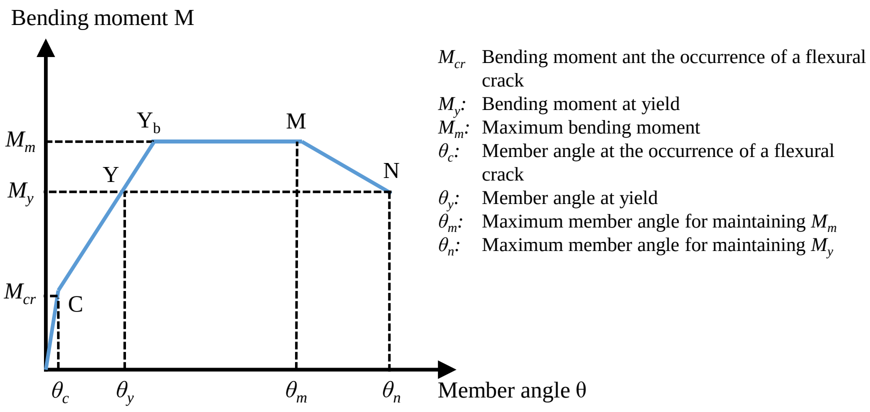
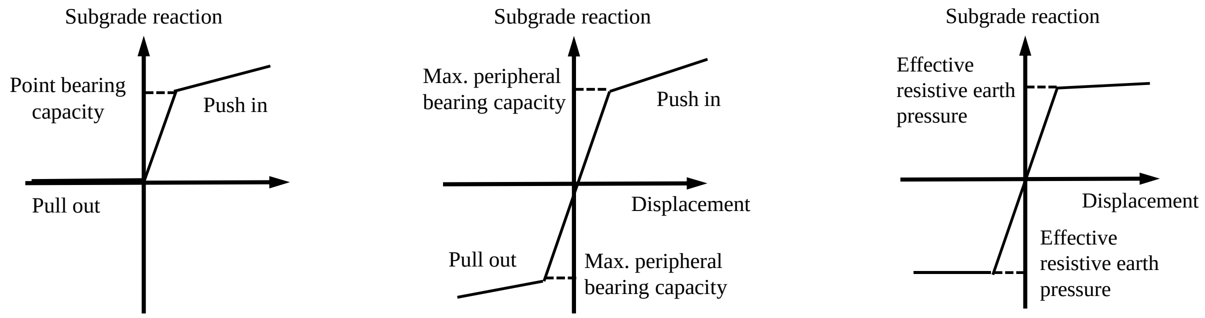


Fig. 7 Nonlinearity of reinforced concrete members[1)]



(a) Vertical spring (b) Frictional springs between soil and piles (c) Horizontal spring

Fig. 8 Nonlinearities of soil springs1)

At first, a push over analysis was conducted to check the failure mode and the damage process of the structure, by gradually increasing horizontal inertial force applied to each mass until the strength of the structure decreased sufficiently. As a result, it was confirmed that the target structure showed bending yielding at footing at first, and strength decreased after yielding of foundation ¿?. In addition, all of the member did not show shear failure during the push over analysis(Fig. 9). From the load-displacement relationship, the yield seismic coefficient was calculareted to be 0.588, which is assumed as shown in Fig. 10 in the seismic standard. Furthermore, the natural period and the yield seismic intensity was evaluated as 1.056 sec by the equation (1). Both the k_{heq} and the T_{eq} were used for obtaining response value of the whole structure due to inertial force by a nonliner sigle degree of freedum model as mentioned later. Fig11 shows a defiition of yiled point of stuctures assumed in the analysis.

$$T_{eq} \cong 2.0\sqrt{\frac{\delta_{eq}}{k_{heq}}} \qquad (1)$$

where T_{eq} : Teequivalent natural period of structure

δ_{eq}: displacement corresponding to turnoff point of entire structure system (m)

k_{heq} : horizontal seismic coefficient corresponding to turnoff point of entire structure system

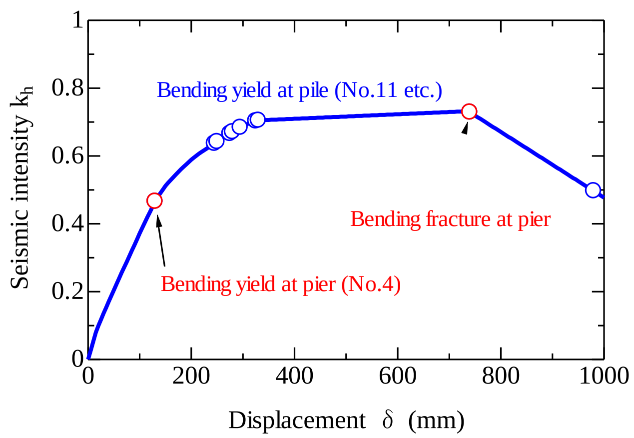


Fig. 9 Check of failure mode

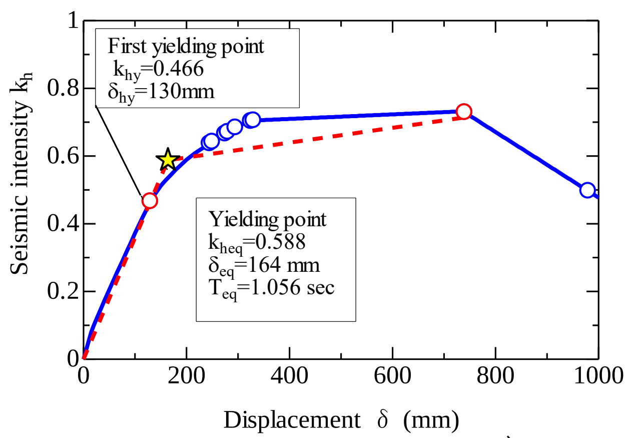


Fig. 10 Determining of the natural period and the yield intensity

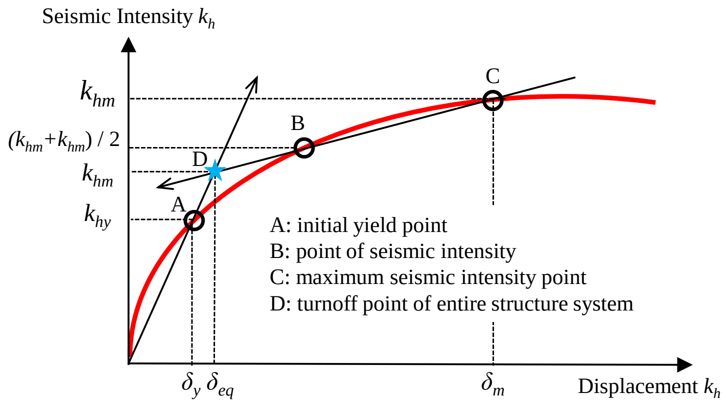
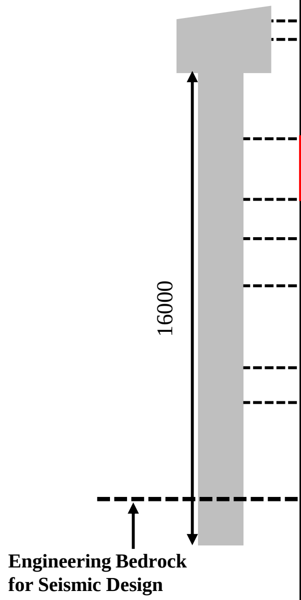


Fig. 11 Yileding point of a structure assumed in design[1)]

The surface ground around the structure was the same ground with that modeled in the ground response analysises described in Chapter 2. Physical and Mechanicla properties of each layers used in the structual analysis were summarized in Fig. 12. As mentined in Chapter 2, seismic responses of surface ground were remarkably influeced by testing method for deteriming nonlinear deformation properties of soils.



Type of Soil	N Value	Layer Thickness (m)	Dry unit weight (kN/m^3)	Cohesion (kN/m^2)	Internal Friction Angle (degree)	Shear Wave Velocity (m/s)
Cohesive	1	0.60	15	15	—	70
Cohesive	2	3.40	15	25	—	110
Toyoura sand	—	2.00	19	—	35.4	150
Sandy	22	1.30	18	—	36.6	135
Sandy	19	1.70	18	—	35.5	135
Cohesive	7	2.75	16	79	—	175
Sandy	33	1.15	18	—	37.8	175
Sandy	30	3.30	18	—	36.4	160
Sand Gravel	70	12.90	20	—	42.5	390

Fig. 12 Target ground (The same ground with Fig. 2)

3.2 Response values of the bridge pier

The "inertial force" and "ground displacement" should be considered as seismic actions. At first, seismic response due only to inertial force was evaluated, followed by due to inertial force and ground displacement. In this analysis, 3 cases were conducted, in which 3 different results of the dynamic ground response analysis were used. In CASE 1 and 2, the dynamic ground response analysis were conducted using nonlinear deformation properties of the Toyoura sand obtained by the conventional and the proposed testing method. The result of the HGRS was used in CASE 3 as the most precise ground response for comparison.

3.2.1 Response values due to internal force

Response of the whole target structure was simply calculated with dynamic analyses with a nonlinear single degree of freedom model(Fig. 13), in which the nonlinearity of the structure was modeled by the Clough model shown in Fig. 14 using k_{heq} and T_{eq} determined in 3.1. Time histories of acceleration at surface obtained from 3 different groud response analyses were applied to the SDPF model, and response ductility ratios were obtained.

Table 1 summarises the calculated response ductility ratios obtained from the reponse analyses. The analytical results clearly shows the response ductility ratios of CASE 1(1.83) and 2(1.85) were almost the same because there was no significant difference in the response acceleration at the surface. The response ductility ratio of CASE 3(2.08) was slightly larger than those of CASE 1 and 2, but the discrepancy of them is not significant.

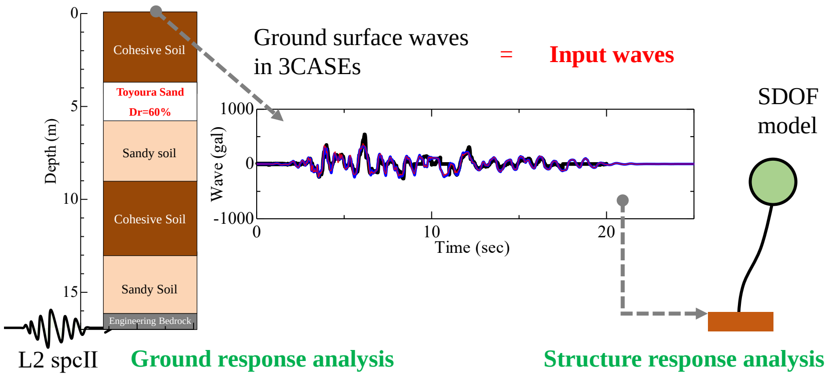


Fig. 13 Outline of determing response of a structure due to inertial force by the nonlinear SDOF model

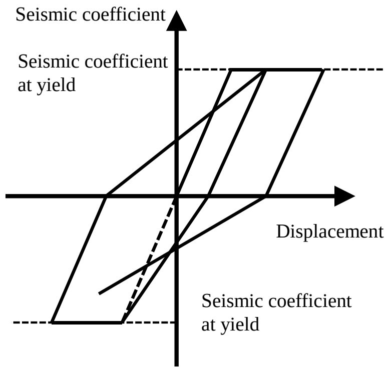


Fig. 14 Skeleton curve assumed in the Clough model[1)]

Table 1 Response ductility ratios

CASE 1: Conventional method	1.83
CASE 2: Proposed method	1.85
CASE 3: HGRS	2.08

Fig. 15 shows the elastic acceleration response spectra at surface obtained from the acceleration time histories used in CASE 2 and 3. As the response value around 1.06 second which was the natural period of the bridge pier were almost the same, each model showed almost the same response ductility ratios.

As mentioned in Chapter 2, relatively large reponse acceleration was observed in the HGRS due to cyclic mobility, which might excite short period accelration between 0.1 to 0.4 second in this case as shown in Fig. 14. As natural period of usual railway structures in Japan is around 0.5~1.0 second, such large acceleration in short period response due to cyclic mobility may not affect dynamic behavior of railway structures, although it cannot be detect by the time domain nonlinear ground response analyasi based on the total stress theory.

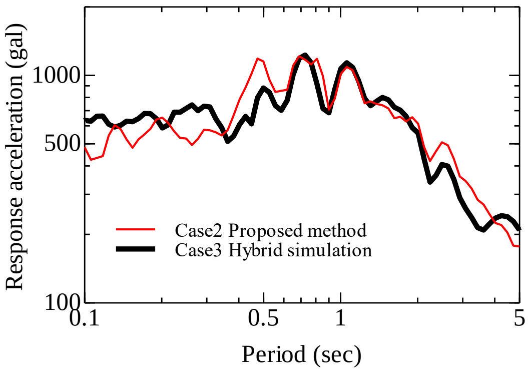


Fig. 15 Acceleration response spectra at ground surface obtained in CASE 2 and 3

3.2.2 Response values due to internal force and ground displacement

Response values of members composing the structure were evaluated considering both intertial force and ground displacement by the response displacement method. Table 2 shows the response values of significantly damaged member, and summarized verification of the performance against the level 2 earthquake. The design limit value of all the members was set as the damage level 2.

In CASE 1, damage level 4 was detected at the pile head. On the other hand, in CASE 2, the damage levels of all the members were restricted within level 2. Damage levels in CASE 3 were almost the same with those of CASE 2, because the ground displacements obtained from both had been almost the same as shown in Fig. 2. In order to satisfy the required peformance of CASE 1 in which the nonlinearlity of the Toyoura sand obtained from the conventional tesing method was used, it was necessary to increase diamter of the axial reinforcement diameter from 32 mm to 35 mm.

These results showed that the ground displacement could significantly affect seismic designs of pile foundations supporitng bridge piers. Furthremore, it is also comfirmed that the determination of shear stiffness at large strain level is very important for seismic designs of deep foundation and underground structures, such as tunnels. As the proposed method could give adequate ground displacement comaring to that of the HGRS, it can contribute to reasonable design for pile foundation of bridge piers as well as underground structures.

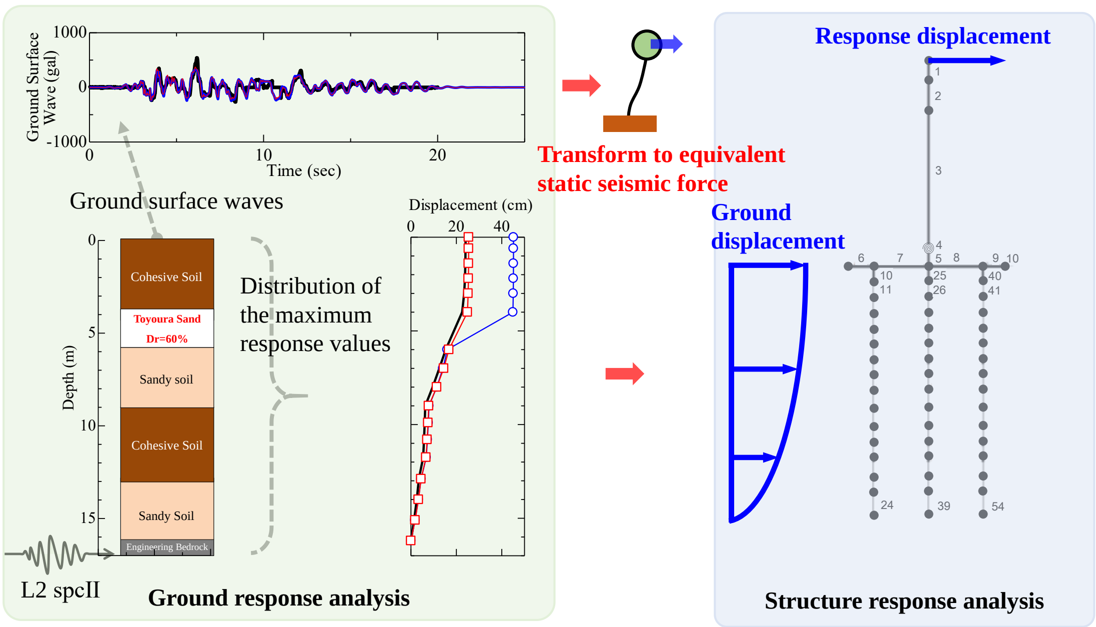
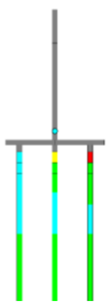
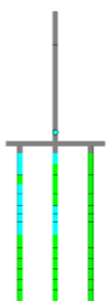
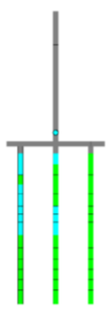
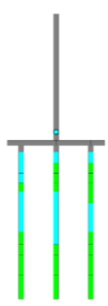


Fig. 16 Outline of determing response of a structure due to inertial force and ground displacement

Table 2 Response values, limit values and verifications

CASE no.	CASE 1	CASE 2	CASE 3	CASE 1
Axial reinforcement (SD390)	24 rebars with a diameter of 32mm			24 rebars with a diameter of 35mm
Equivalent natural period	1.06	1.06	1.06	1.05
Yield seismic coefficien	0.59	0.59	0.59	0.59
Response ductility ratios	1.83	1.85	2.08	1.85
Member site	Right pile head	Central pile head	Central pile head	Right pile head
Shear force Vd (kN)	2845	3152	3004	4547
Shear capacity Vvd (kN)	5887	5396	5423	5953
Structural fracture mode	Bending	Bending	Bending	Bending
Response curvature φd (1/m)	-0.02464	-0.00485	-0.00485	-0.0159
Design limit value of curvature for damage level 1φyd (1/m)	-0.00543	-0.00412	-0.00412	-0.00544
Design limit value of curvature for damage level 2φyd (1/m)	-0.02079	-0.02169	-0.0217	-0.02087
Design limit value of curvature for damage level 3φyd (1/m)	-0.02079	-0.02999	-0.023	-0.02087
γi·φd/φmd	1.19	0.22	0.22	0.76
Response damage level	4	2	2	2
Limit damage level	2	2	2	2
Damage level distribution Explaantory note Rigid Zone ----------- Damage level 1 ------ Damage level 2 ------ Damage level 3 ------ Damage level 4 ------				

4. Conclusions

The authors have proposed a new testing method to determine G/G_0-γ and h-γ relationships necessary for the dynamic ground response analysis against a large-scale earthquake. The past study clearly showed that the dynamic ground response analysis with the G/G_0-γ and h-γ curves obtained from the proposed method could produce almost the same seismic ground response with that of the HGRS which could give the most accurate seismic ground response, although the inertial force might be evaluated slightly smaller. On the other hand, it was also confirmed that the dynamic ground response with conventional testing method might give extremely larger ground displacement at ground surface. Trial calculations for seismic response values of a bridge pier supported by a pile foundation were conducted to verify the applicability of the proposed soil testing method. As a result, the following knowledges were obtained.

1) The time domain nonlinear ground response analyasi based on the total stress theory cannot evaluate quick increase of acceleration with short period(approximately less than 0.4 seccond) due to cyclic mobility even though nonlinear deformation properties of soils are obtained from the proposed testing method. As natural period of usual railway structures in Japan is around 0.5~1.0 second, such quick increase of acceleration due to cyclic mobility would not affect dynamic behavior of railway structures.
2) In the case of the conventional method(CASE1), damage level 4 was observed at the pile head. On ther other hand, the damage levels of all the members were restricted within level 2 in the case of proposed method(CASE2). In order to satisfy the required peformance of CASE 1 in which the nonlinearlity of the Toyoura sand obtained from the conventional tesing method was used, it was necessary to increase diamter of the axial reinforcement diameter from 32 mm to 35 mm.
3) Determinations of nonlinear defromation properties of soils at large strain level is very important for seismic design of deep foundation, underground structures and so on. The proposed method can give adequate ground displacements and contributed to reasonable design for such structures.

5. References

[1] Railway Technical Research Institute, “Design Standards for Railway Structures and Commentary (Seismic Design),” Maruzen company, 2012. (in Japanese)

[2] Yuta NOGAMI, Yoshitaka MURONO: Nonlinear Hysteresis Model Taking into Account S-shape Hysteresis Loop and its Standard Parameters, Proceedings of 15th World Conference on Earthquake Engineering.

[3] Nozomu YOSHIDA. Takeo MIKAMI: Establishment of Cyclic Shear Deformation Characteristics Test that Responds to Demands of Times, Japanese Geotechnical Society 58(2), 1-5, 2010-02-01. (in Japanese)

[4] Jun IZAWA, Yoshitaka MURONO: A Fundamental Study on Soil Laboratory Testing method for Nonlinear Seismic Ground Response Analysis, International Mini Symposium CHUBU, pp.147-154, 2016.

[5] Akira SUZUKI, Jun IZAWA, Akihiro TOYOOKA, Kenichi KOJIMA: Construction of an Appropriate Response Evaluation Method for the Surface Ground in Seismic Design, RTRI REPORT Vol.32, No.9, Sep.2018. (in Japanese)

[6] Jun IZAWA, Akihiro TOYOOKA: Evaluating seismic behaviors of surface ground - Application of hybrid ground response test -, Japan Society of Civil Engineers Magazine, Vol.103, No.12, 2018. (in Japanese)

[7] Akihiko NISHIMURA, Yoshitaka MURONO: Evaluation of Non-linear Earthquake Response of Bridges using Strength Demanded Spectra, RTRI REPORT Vol.13, No.2, 1999.2. (in Japanese)

