Seismic response analysis and damage verification of notojima bridge during noto peninsula earthquake

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SEISMIC RESPONSE ANALYSIS AND DAMAGE VERIFICATION OF NOTOJIMA BRIDGE DURING NOTO PENINSULA EARTHQUAKE

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ABSTRACT : Noto Peninsula earthquake of magnitude 6.7 occurred near Noto Peninsula in Japan on March 25, 2007. Notojima Bridge across Nanao Bay, which was completed in 1982, is located about 30 km east-southeast of the epicenter. It is 1050 m long multi-span bridge consisting of 21 spans, in which the 10 and 8 spans are simply supported PC girder bridges and the central three spans are a rigid frame PC bridge with pin-connection at the mid-span. Notojima Bridge sustained considerable damage in many RC piers, bearing supports and expansion joints. Especially the piers of P10 and P13 in the central portion of the bridge sustained damage asymmetrically in spite of the symmetrical figure of the superstructure and piers. In order to verify unexpected damage, the central portion is investigated based on seismic response analysis taking account of the inelastic hysteretic property of piers and the strong-motion data observed near the bridge. It is found that the difference of steel pipe piles between P10 and P13 might affect the asymmetrical damage.

KEYWORDS: Noto Peninsula earthquake, Notojima Bridge, Rigid frame PC bridge, Seismic response analysis, Earthquake damage

1. INTRODUCTION

The magnitude 6.7 earthquake occurred near Noto Peninsula in Japan on March 25, 2007 at 00:41:57 UTC. The completely destroyed houses, which were older houses constructed of wooden frame, amounted to 582. Many soil slope failures and falls of rock were occurred, and destroyed roadways. A lot of bridges in the area sustained minor damage such as the displacement of girders, the deformation of bearing supports and the shear flow of backfilling soil behind abutments, however they didn't need to be closed after the temporary inspection.

Notojima Bridge, which crosses Nanao Bay, has been 26 years since the completion. It is located about 32 km east-southeast of the epicenter. It is 1050 m long multi-span bridge consisting of 21 spans, in which the 10 and 8 spans are simply supported Prestressed Concrete (PC) girder bridges and the central three spans are a rigid frame PC box girder bridge with pin-connection at the mid-span. The seismic performance was verified by static method based on the seismic coefficient method [1], in which the design horizontal seismic coefficient was adopted as 0.19 being equivalent to the inertia force owing to the maximum horizontal acceleration of 186 gal. After the Hyogo-ken Nanbu Earthquake in 1995 the earthquake-resistant work had been carried out for the major bridges in Japan, and the aseismic work to Notojima Bridge was starting just before the earthquake but too late.

Though the maximum horizontal acceleration at the observation point (ISK007) about 5 km south-southeast of the bridge was only 209 gal, Notojima Bridge sustained considerable damage in many Reinforced Concrete (RC) piers, bearing supports and expansion joints. Especially the piers of P10

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and P13 in the central portion of the bridge sustained damage asymmetrically in spite of the symmetrical figure of the superstructure and piers. The objective of this paper is to confirm its damage by means of seismic response analysis taking account of the inelastic hysteretic property of piers and the strong-motion data observed at ISK007.

2. STRUCTURE PROFILE AND EARTHQUAKE DAMAGE OF NOTOJIMA BRIDGE

Figure 1 shows the area of Noto Peninsula, the location of the epicenter, the strong-motion observation stations (ISK005, ISK006 and ISK007) and the Notojima Bridge. The amplitude diagrams of the N-S and E-W components in each station are also shown in Figure 1. Furthermore the maximum accelerations calculated by composing of three vectors N-S, E-W and U-D are 903, 945 and 221 gal in the stations of ISK005, ISK006 and ISK007, respectively.

Figure 2 shows the side view of Notojima Bridge. The damage levels in piers, which are indicated with B, C or D, have been judged by visual inspection. The level B corresponds to the reinforcement

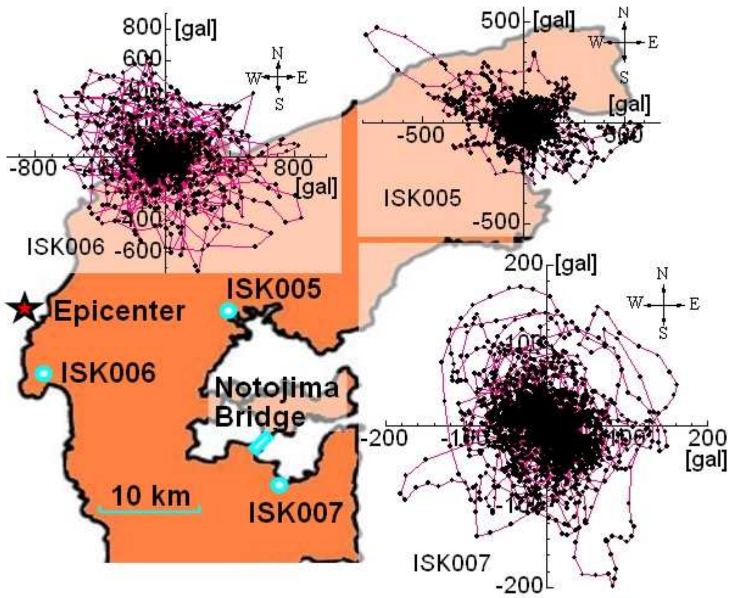


Figure 1. Amplitude diagram of the strong-motion data observed in ISK005, ISK006 and ISK007

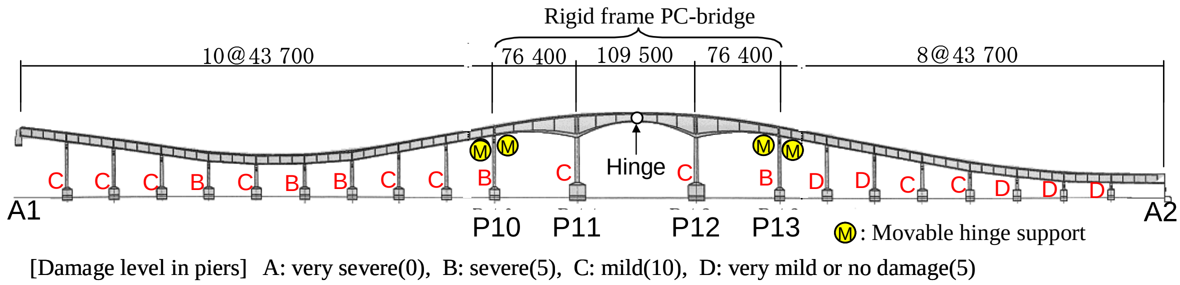


Figure 2. Side view of Notojima Bridge and Damage level in piers

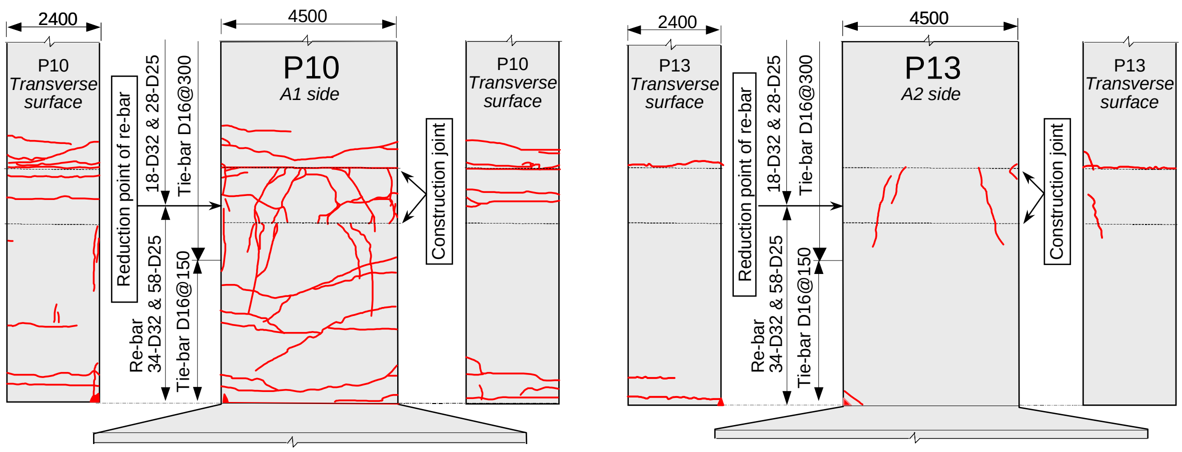


Figure 3. Crack distribution in piers P10 and P13

break and buckling and/or to the stripping and cracks of cover concrete. The level C corresponds to the local stripping and cracks of concrete covering. The piers on A1 (Notojima Island) side sustained relatively severe damage. Indeed the bridge is longitudinally asymmetric, but the rigid frame PC bridge of central three spans is symmetric. The damages of both piers P10 and P13 were evaluated as the level B, however the crack density of P10 is obviously higher than that of P13 (see Fig.3).

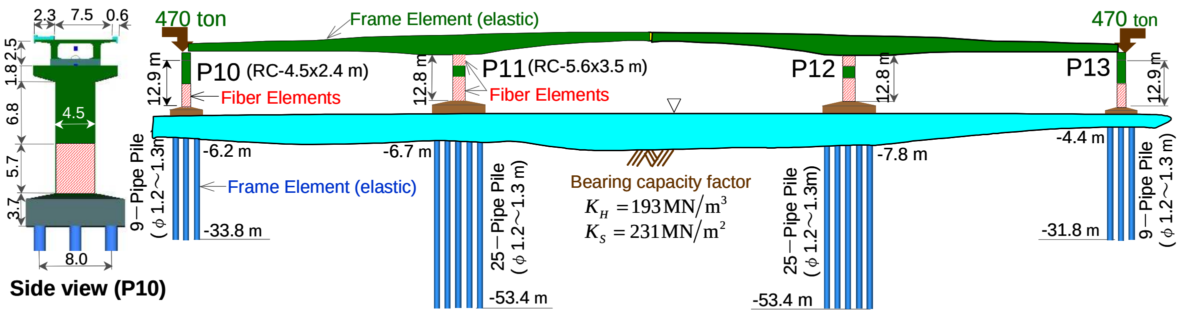


Figure 4. 3-spans continuous rigid frame PC bridge

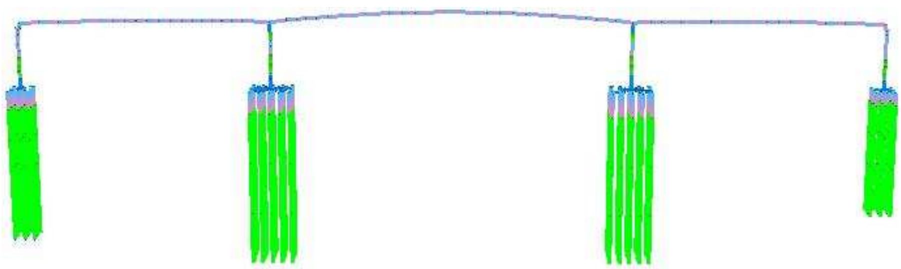
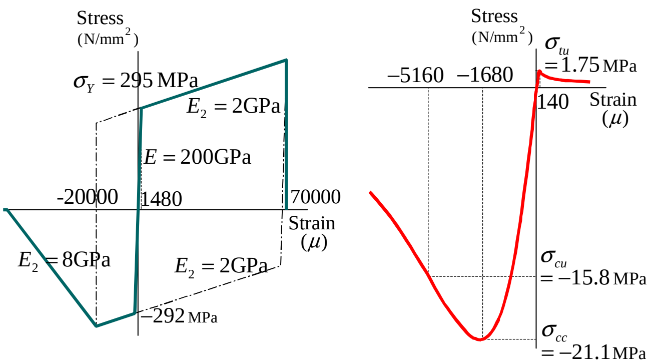


Figure 5. Analytical model

3. NON-LINEAR SEISMIC RESPONSE ANALYSIS

In this paper a non-linear seismic response analysis was conducted for the 3-span continuous rigid frame PC bridge in order to confirm the damage level inspected visually. The FEM computer code, UC-win/FRAME(3D) [2], for simulating the 3D non-linear behavior of this bridge under seismic load was used. Figure 4 shows an outline of the object bridge, and also Figure 5 shows FE-Model in which elastic 3D frame elements were basically adopted. The rigidities and mass of each element in the superstructure were calculated in consideration of an asymmetrical cross section. Furthermore, the concentrated mass of the half dead load of the adjoining span was applied on the top of P10 and P13 vertically and transversely, because of the support condition of moving in longitudinal direction. The multi-piles are made of steel pipe and the underwater pile lengths vary from 4.0 to 7.4 m in piers. Each pile was modeled by elastic 3D frame elements which were supported with the distributed spring of the bearing factor K_s in consideration of the N-value of the ground varying within 20 to 40. Fiber model was used for the damage assessment of Reinforced Concrete (RC) piers. It was applied for piers P10 and P13



(a) Reinforcement (b) Concrete

Figure 6. Non-linear properties used for fiber model elements

from the footing to the reduction point of a longitudinal reinforcement and also used on the both sides of piers P11 and P12. Figure 6 shows the non-linear material properties for fiber model elements.

Horizontal acceleration and spectrum at the observation point ISK007, which are located in south-southeast of the bridge, are shown in Figure 7. The frequencies of 0.7Hz and 1.3-1.5 Hz in the longitudinal direction and those of 0.8 Hz, 1.0Hz and 2.6Hz in the transverse direction are shown in the spectrum.

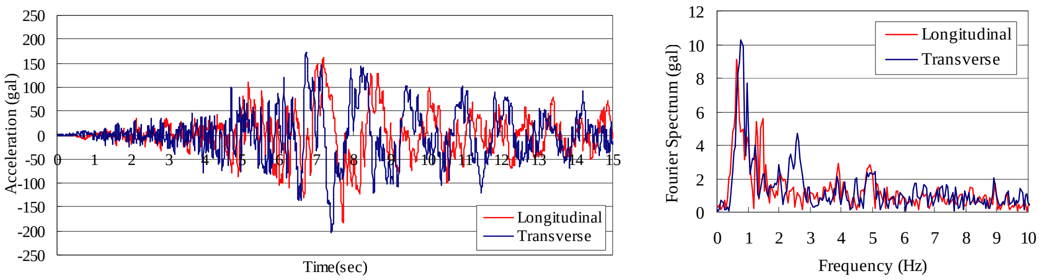


Figure 7. Horizontal acceleration and spectrum at the observation point ISK007

4. VIBRATION CHARACTERISTICS

Eigen value analysis was carried out to confirm the natural frequencies and vibration modes. The results of the natural frequencies, participation factor and vibration modes are shown in Figure 8. It shows that the asymmetric mode is occurred by influence on the length of the steel pipe piles in P10 and P13. Therefore the mode amplitude of the P10 in 1st vibration mode is larger than that of the P13. Vibration test by the ambient vibration was carried out in the P10 and P13. The frequency of the P10 and P13 in transverse direction was 1.3Hz. The validity of the bridge model is confirmed with comparison between the results of experiment and analysis.

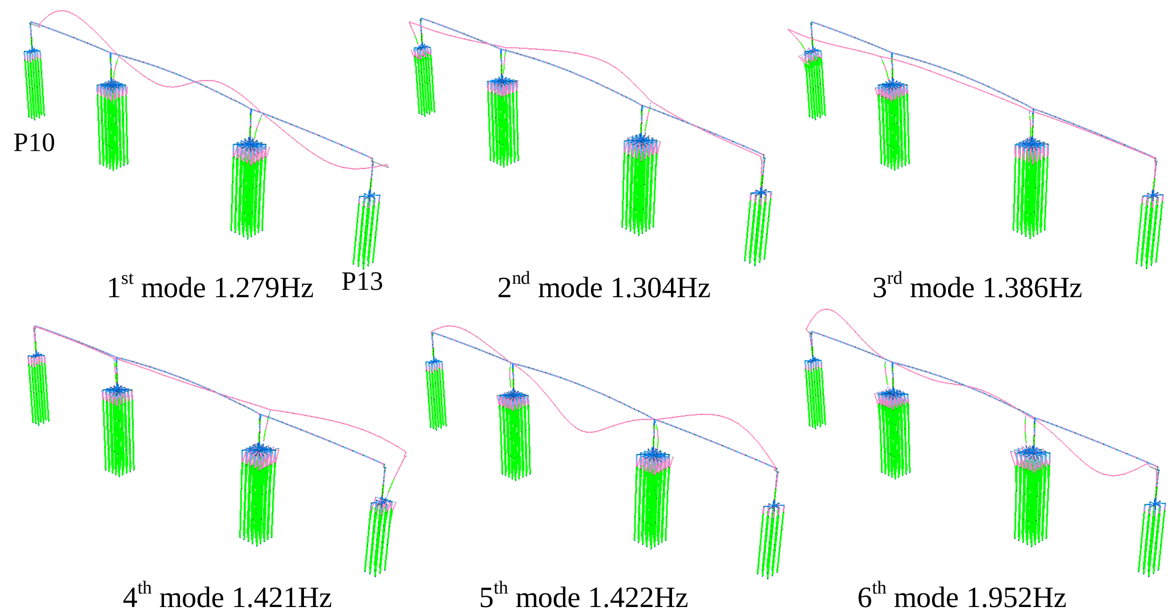
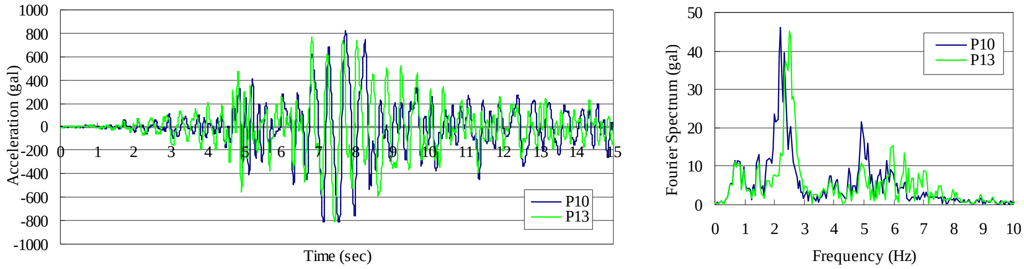


Figure 8. Frequencies and vibration modes

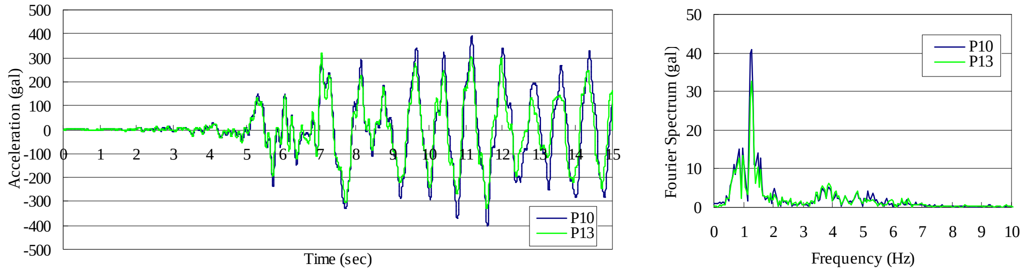
Damping constant (*h*) obtained by ambient vibration using Random Decrement (RD) method [3] was 0.03. Damping in the analysis was assumed Rayleigh damping. Damping constants of the superstructure and piles were used 0.03 and 0.10, respectively. And also damping constants of the piers were used 0.03, 0.04 and 0.05 as parameters considering the amplitude of seismic force.

5. RESULTS OF NON-LINEAR SEISMIC RESPONSE ANALYSIS

Horizontal acceleration and spectrum at the top of P10 and P13 obtained by non-linear analysis (*h*=0.03) are shown in Figure 9. The vibration modes, which have big participation coefficients in longitudinal and transverse direction, are shown in the analytical spectrum. The acceleration amplitude of transverse direction at the top of P10 is larger than that of P13.



(a) Longitudinal direction



(b) Transverse direction

Figure 9. Horizontal acceleration and spectrum at the top of P10 and P13 by non-linear analysis

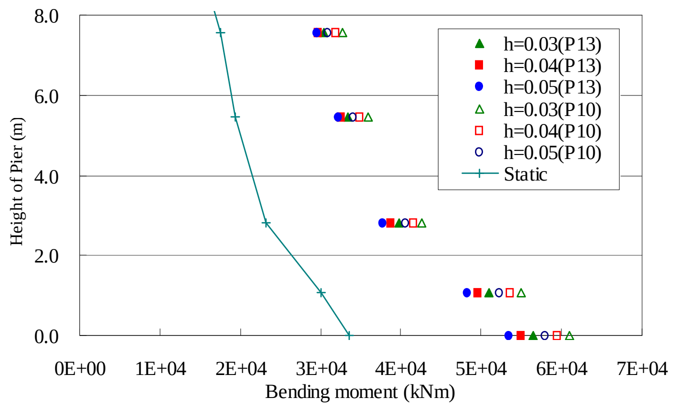


Figure 10. Bending moment diagram

The bending moments of P10 and P13 by the static and dynamic seismic loads are shown in Figure 10. This figure also shows the result of along height of the pier using each damping constant. The damping constants give minor influence on the bending moments. The results of P10 are larger than that of P13. Dynamic amplification at the bottom of the P10 was 1.7 to 1.8 for the static seismic load.

Figure 11 shows the analytical damage (*h*=0.03) of the cross section in P10 and P13. The each contour shows the state of damages in the fiber elements. The damage area of P10 is larger than that of P13. It is found that the difference of steel pipe piles between P10 and P13 might affect the asymmetrical damage.

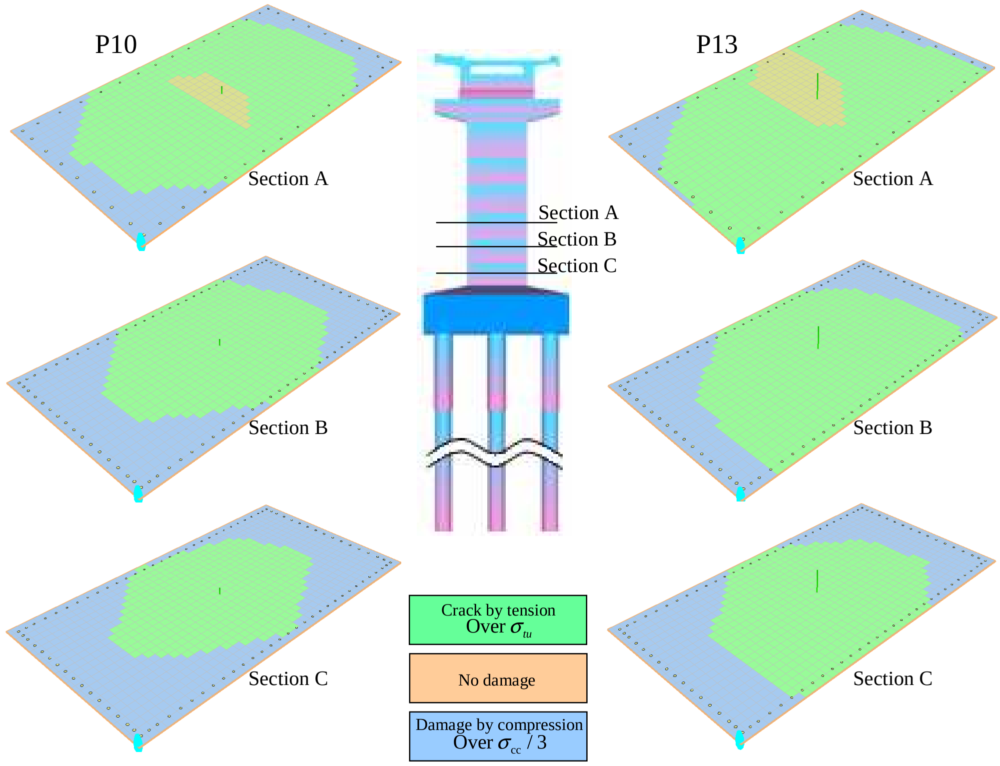


Figure 11. Analytical damage of the cross section in P10 and P13

6. CONCLUSIONS

The conclusions drawn from this study are shown as follows.

1. The damages of P10 and P13 in 3-spans continuous rigid frame PC box girder bridge with pin-connection at the mid-span were evaluated by the visual inspection. The crack density of P10 is obviously higher than that of P13.

2. It is clear that the vibration mode and the seismic behavior of the rigid frame PC box girder bridge by 3D eigen value analysis and 3D non-linear seismic analysis, respectively.

3. The analytical damage area of P10 is larger than that of P13. It is found that the difference of steel pipe piles between P10 and P13 might affect the asymmetrical damage.

ACKNOWLEDGEMENTS

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