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Title: Forces acting on bridge abutments over liquefied ground

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- 8 Abstract:

9 Large earthquake-induced displacements of a bridge abutment can occur, when the bridge is built 10 on a floodplain or reclaimed area, i.e., liquefiable ground, and crosses a water channel. Seismic 11 responses of a bridge abutment on liquefiable ground are the consequence of complex interactions between the abutment and surrounding soils. Therefore identification of the factors dominating the 12 13 abutment response is important for the development of simplified seismic design methods. This paper presents the results of dynamic three-dimensional finite element analyses of bridge abutments 14 adjacent to a river dike, including the effect of liquefaction of the underlying ground using 15 16 earthquake motions widely used in Japan. The analysis shows that conventional design methods 17 may underestimate the permanent abutment displacements unless the following two items are 18 considered: (1) softening of the soil beneath the liquefiable layer, due to cyclic shearing of the soil 19 surrounding the piles, and (2) the forces acting on the side faces of the abutment.

- 20
- 21 Keywords:
- 22 abutment, pile foundation, liquefaction, earth pressure, finite element analysis
- 23

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- 28 1. Introduction
- 29

Large earthquake-induced displacements of a bridge abutment can occur, when the bridge is built on a floodplain or reclaimed area, i.e., liquefiable ground, and crosses a water channel. When the abutment is located adjacent to a river dike or revetment (See Fig. 1), it is especially prone to movement toward the waterfront during an earthquake, when the underlying ground liquefies.

In conventional Japanese design methods, the seismic performance of bridge abutments on liquefiable soil is assessed by pushover analysis, i.e., by calculating the response of the abutment subjected to (1) the inertial force of the superstructure and the abutment, and (2) the seismic active earth pressure of the backfill. The kinematic load induced by interaction between the surrounding soil and structure has been neglected, for simplicity, because liquefaction of the foundation soil appears to cause a reduction of lateral soil resistance [1].

40 For structures subjected to lateral spread on level ground or gentle slopes, many procedures that 41 account for effects of the lateral spreading on their performance have been proposed [2][3][4][5]. 42 However the existing procedures may not be directly applicable to bridge abutments since the 43 seismic performance of abutments is also affected by many factors, such as, (1) local deformation 44 of the adjacent river dike, (2) deformation of foundation soils caused by the weight of a road 45 embankment connected to the abutment, and (3) slumping of the road embankment itself. Because the seismic response of a bridge abutment on liquefiable ground is the consequence of complex 46 47 interactions among all these factors, identifying the factors dominating the abutment response is 48 critical when simplified seismic design is used.

49 In addition, the critical moment for the abutment during earthquakes has to be carefully chosen 50 for such a simplified seismic design method: When the seismic performance of a pile-supported 51 bridge abutment is assessed with the pushover analysis, the response coinciding with the arrival of 52 the largest (principal) shock is generally assumed to represent the critical design condition for the 53 abutment. However, the horizontal displacement of the abutment coinciding with the principal 54 shock is not necessarily the maximum displacement the abutment will experience and the same is 55 also true for the strength and ductility demands for the structural members that form the bridge 56 abutment.

57 This paper presents the results of dynamic three-dimensional finite element analyses on bridge 58 abutments built on liquefiable ground adjacent to river dikes, taking into account the effect of 59 liquefaction on abutment response. Two configurations of the surrounding ground were modelled, 60 and the seismic response of abutments both with and without piles is presented. Forces acting on 61 the abutment as well as calculated responses of the abutment are described in detail to demonstrate 62 (1) the critical moment assumed in the conventional design methods, i.e., pushover analyses, being 63 not always appropriate and (2) marked contribution of the forces acting on the side faces of the 64 abutment in the forces acting on the abutment. The results of the analyses are presented, including 65 recommendations for revisions in the current conventional design methods employing the pushover 66 analysis for bridge abutments.

67

68 2. Numerical analysis overview

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70 Highway bridge abutments constructed near a river channel were modelled and analyzed. 71 Two configurations of the ground surrounding the abutment were considered; a plan view and cross 72 section of the models is illustrated in Fig. 2. The bridge crosses a river, with a distance between 73 abutments of 70m. For a fully coupled three-dimensional finite element analysis, limited 74 computer resources did not permit the use of a fine mesh, so a relatively coarse mesh was employed 75 (cf. Fig. 6 in the following section). Because of this limitation, the pile foundation was modelled by relatively small number of piles having large diameter.; The pile foundation was modelled with 76 77 six $D=2m \ge L=20m$ cast-in-place concrete piles, arranged in 2x3 grids with 5m spacing. The 78 width of the bridge is 15m, and the slope of the river dike and road embankment connected to the 79 10m-high pile-supported abutment is 1V:2H. The water level was set at 7.5m below the crest of 80 the dike. The original ground level is 5m above riverbed for the cases illustrated in Fig. 2(a), and 81 10m for the cases shown in Fig. 2(b). The thickness of the liquefiable layer (loose sand deposit) 82 below the water table is 12.5m, and the materials of the dike and embankment are assumed to be the 83 same as that of the surface layer that lies above the water table. The piles are installed into a 84 bottom non-liquefiable layer (dense sand deposit) through the liquefiable layer.

85 In the numerical analyses, only a half-width of the road bridge in the y-direction was modelled, in order to take advantage of symmetry. The width of the analytical domain in y-direction is 100m, 86 87 the length in x-direction is 240m, and the depth in z-direction from the riverbed is 20m. The 88 x=-40m and x=200m planes were allowed to move freely in the x- and z-directions, but not in the v-direction. At z=-20m, all movement was restrained. Fluid flow velocities normal to the 89 90 analytical domain boundary were set to zero at the side and bottom boundaries. Soils, stem and 91 base of the abutment were modelled by solid elements, and the piles were modelled with elastic 92 beam elements. Because Potyondy [6] reported that the wall friction angle between smooth 93 concrete and sand is around 75% of the soil shearing resistance, no interface element was inserted 94 between the abutment and the adjacent soil. The extended sub-loading surface model proposed by 95 Hashiguchi & Chen [7] was adopted for the soil layers. Material parameters of the soil layers and

96 the structural components are listed in Table 1, where Gs=specific gravity, e_0 =initial void ratio, λ & 97 κ =slopes of the compression & swelling lines in v-ln(p') plane, v=Poisson's ratio, k=hydraulic 98 conductivity, ρ =unit density, E=Young's modulus, I=flexural rigidity of the beam, A=area of beam 99 section, and the other parameters are specific parameters for the constitutive model used. 100 Liquefaction resistances of the non-liquefiable and liquefiable layers are summarized as the cyclic 101 shear stress ratio plotted against the number of loading cycles that cause liquefaction in triaxial tests, 102 as shown in Fig. 3. Typical stress path and stress-strain relations for the liquefiable layer soil are 103 shown in Fig. 4. Details of the numerical analysis code are described in Takahashi [8].

104 To evaluate the seismic performance of structures, two levels of earthquake motions are used 105 in Japan [9]; one is a moderate earthquake motion whose return period is smaller than the life time 106 of a structure (from 50 to 100 years) and the others are strong earthquake motions whose return 107 period is more than several hundred years. For the latter, two types of earthquake motions are 108 considered; one is the inland earthquake motion and the other is the subduction earthquake motion. 109 In this study, numerical analyses were performed to assess the seismic performance of the bridge 110 abutments against the strong earthquakes. Figure 5 shows the applied earthquake motions. 111 These are widely used in practice in Japan. One is the inland earthquake motion (Fig. 5(a), sample waveform for Spectrum II [10].) The other one is the subduction earthquake motion (Fig. 5(b), 112 113 sample waveform for Spectrum I [10].) The earthquake motion is applied in the bridge axis 114 direction (x-direction). The equations of motion are integrated using Newmark's β method, with 115 a time step $\Delta t=0.005$ sec. System damping is represented by Rayleigh damping using a damping 116 ratio of 2.5 % in a first mode of free vibration of the system.

117 Six calculations were conducted, as listed in Table 2. Except for P-H(EQ1), the inland 118 earthquake motion (Fig. 5(a)) was applied. Before performing a dynamic response analysis, a 119 geostatic analysis was conducted, by applying the self weight gradually, in order to establish the 120 initial stress state of the system. In P-L, the road embankment was connected to the pile-supported abutment, with the original ground level set to a height of 5m above the riverbed (Fig. 2(a).) For 121 122 this case, waterward and landward stretching of the river dike, and the slumping of the road 123 embankment, were expected. In P-H, because the ground level was set to a height of 10m above 124 the riverbed (at the same level of the roadway, Fig. 2(b),), the deformation mode of the surrounding 125 ground was thought to be simpler than for case P-L, since no complicated spreading mode of the 126 foundation ground, induced by the slumping of the road embankment in a direction perpendicular to 127 the bridge axis, was expected. By comparing cases P-L and P-H, the effect that the geometry of 128 the surrounding ground has on the seismic response of the abutment can be observed.

129 In case P-H(Fix), the "strut effect" of the bridge deck on abutment response was modelled by

130 constraining the horizontal displacement of the top of the abutment. In the geostatic analysis, it 131 was assumed that the top of the abutment is initially in contact with the deck before shaking 132 commences for all the cases. Thus, the initial stress state of the system for P-H(Fix) is the same as 133 that for the case without the strut effect (P-H). Since the critical moment in the abutment may 134 vary depending upon the earthquake motion waveform, the subduction earthquake motion (Fig. 135 5(b)) is applied in P-H(EQ1). For comparison, responses of the abutment without piles are 136 calculated in NP-L and NP-H.

137 The limitations of the analysis are: (1) the bridge deck and intermediate piers were not 138 modelled, and (2) the top of the abutment was free to move, without restriction, except in case P-H(Fix). In other words, the dynamic interaction between the deck and the abutment was not 139 140 modelled, because our main concern was the interaction between the abutment, foundation ground, road embankment, and river dike. As a result of these limitations, the calculated displacements 141 142 and vibration modes of the abutment may be different from those in a real bridge-foundation system. 143 In addition, since the only the two waveforms are considered, the observation on numerical analyses in this study has to be taken as one of the examples. Even though we label one of the input 144 145 motions as the inland earthquake motion and the other as the subduction earthquake motion, the used motions are not necessarily representative waveforms for these types of motions. 146

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148 3. Observations on numerical analyses

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In conventional Japanese design methods, seismic responses of an abutment are typically calculated by the pushover analysis assuming that (1) the response coinciding with the arrival of a principal shock represents the critical design condition for an abutment and (2) effects of the lateral spreading on the abutment response are negligible. In this section, these major assumptions in the conventional design methods are examined through observations on numerical analyses.

155

156 *3.1. Permanent deformation of the ground surrounding the abutment*

Ground deformations around the pile-supported abutment (P-L & P-H) at t=30sec (when ground shaking almost ceases) are drawn in Fig. 6. Large shear deformation of the loose sand deposit under the abutment and the connected road embankment (the roadway for P-H) was observed in both cases. In the case where the original ground level was lower than the height of the river dike crest (P-L), the road embankment showed relatively large settlements, with marked uneven settlement occurring just above the heel of the abutment pile cap. In addition, the shear deformation of the liquefiable layer under the road embankment is somewhat smaller than for case 164 P-H.

The waterward horizontal displacements of the river dike far from the abutment are not affected 165 by abutment movement. The higher the original ground level, the larger the waterward permanent 166 horizontal displacement of the dike, i.e., the lateral spread. On the other hand, the deformation of 167 the rive dike near the abutment are restrained by the abutment. Due to this restriction, the backfill 168 169 behind the abutment shows permanent horizontal displacement normal to the bridge axis (see the 170 horizontal ground displacement at the river dike's crest level, the lower graphs in Fig. 6). 171 Although the plots for the abutments without piles (NP-L & NP-H) are not shown here, they are 172 approximately the same as those for the pile-supported abutment, except for abutment tilting (large 173 tilting of the abutment was seen in NP-L & NP-H) and relative horizontal displacement between the 174 dike and the abutment (the relative displacement was not large in these cases).

175

176 *3.2. Excess pore water pressure response*

177 Although the above-mentioned permanent deformations of the abutment and surrounding 178 ground were caused by the generation of the excess pore water pressure in the loose sand deposit, 179 this does not necessarily mean that full liquefaction took place over the entire area. In P-L, the 180 loose sand just below the dike and embankment was not fully liquefied, whereas full liquefaction 181 did occur in other areas, as shown in Fig. 7. There are two possible reasons for this: one reason is that the embankment (including the river dike) load on the foundation ground induced shear stress 182 183 in the foundation soil, increasing its strength against cyclic shear stress [11]. The other reason is 184 that the smaller overburden pressure at the original ground surface limits the excess pore water 185 pressure that can develop in the liquefiable layer, resulting in water migration from the embankment 186 foundation toward the original ground, thereby reducing the excess pore water pressure in the 187 embankment foundation.

The liquefiable layer under the roadway is also liquefied in P-H, since it does not meet the 188 189 conditions mentioned above, i.e., unlike P-L, there is no initial shear stress in the plane normal to 190 the bridge axis and no ground water migration from the embankment foundation toward the original 191 ground. For the cases on the non-pile-supported abutment (NP-L & NP-H), the excess pore water 192 pressure ratio just below the abutment is very small, compared to the pile-supported abutment, due 193 to the large confining pressure induced by the abutment weight (for the pile-supported abutment 194 cases, the weight of the abutment is supported by piles, and does not contribute to increasing the 195 confining stress of the liquefiable soil underneath the abutment.)

196

197 *3.3. Horizontal abutment displacement*

198 Figure 8 shows the time histories of the horizontal displacement of the top of the abutment for 199 all cases except P-H(Fix) (the horizontal displacement of the abutment top is constrained in that 200 case), together with the time histories of the channel-side shoulder of the dike far from the abutment 201 (x=-5m, y=100m & z=10m), shown with a dashed line. Negative values of displacement represent 202 waterward abutment movement. When the inland earthquake is applied, the horizontal 203 displacement of the abutment increases with shaking; two large fluctuations in the horizontal 204 displacement time histories coincide with the two large shocks in the input motion (see Fig. 5). 205 For the subduction earthquake, P-H(EQ1), the displacement time history does not show a large 206 jump during the principal shock, and the displacement increases monotonically with the shaking. 207 The permanent displacements of the pile-supported abutments (P-L & P-H) are smaller than those 208 of the abutments without piles (NP-L & NP-H), and the higher the original ground level (for the 209 cases with the surrounding ground geometry shown in Fig. 2(b)), the larger the abutment horizontal 210 displacement. For all the cases the permanent abutment displacement is larger than the 211 displacement that occurs at the time of the principal shock.

212 The horizontal displacements of the river dike far from the abutment are not affected by 213 abutment movement (cf. (1) P-L & NP-L and (2) P-H & NP-H) as mentioned before. The higher 214 the original ground level, the larger the permanent horizontal displacement of the dike, i.e., the 215 lateral spread. The larger resistance of the abutment foundation against spreading results in a larger relative displacement between the abutment and the spreading ground. In the pile-supported 216 217 abutment cases, the relative displacement is large, whereas it is very small for the abutments 218 without piles. Restraint of the ground around the abutment against spreading causes the forces 219 acting on the abutment to be larger, and affects the strength and ductility demands that are made on 220 the structural members.

221

222 *3.4. Earth pressure acting on abutment back face*

223 Figure 9 shows the time histories of the coefficient of horizontal earth pressure acting on the 224 abutment back face (x=5m). The coefficient is defined as an average ratio of the horizontal 225 effective stress to the vertical effective stress. Before the arrival of the principal shock, the earth 226 pressure acting on the back face of the abutment decreases slightly with increasing abutment 227 displacement. The earth pressure time histories show sharp spikes (the maximum earth pressures 228 that occurred during the entire shaking history) coinciding with the arrival of the first principal 229 shock. After the first principal shock, the earth pressure decreased to a nearly constant level when 230 subjected with the inland earthquake motion; when subjected to the subduction earthquake 231 (P-H(EQ1)), the maximum earth pressure does not coincide with the principal shock.

232 In Fig. 9, the coefficients calculated by the Mononobe-Okabe method are shown with arrows. 233 For the cases without piles (NP cases) and the pile-supported abutment case with a lower original 234 ground level (P-L), the maximum values are approximately equal to the Mononobe-Okabe values, 235 while in the P-H cases, especially when constrained against horizontal displacement (P-H(Fix)), the 236 maximum values were larger than the Mononobe-Okabe. In the former cases, the earth pressure 237 coefficient following the principal shock is smaller (close to the active earth pressure coefficient), 238 whereas it is relatively large in the latter cases. Even in the case when the top of the abutment is 239 restrained (P-H(Fix)), the values are far below potential passive earth pressures.

240 These observations are linked to the relative displacements between the abutments and the 241 spreading ground. When either (1) the abutment foundation has a small amount of restraint 242 against movement toward the waterfront (NP cases), or (2) the lateral spread is relatively small because of a lower original ground level (P-L), the relative displacement is small and the 243 244 spread-induced component of earth pressure is also small. In these cases, the peak earth pressure 245 can be approximated by the Mononobe-Okabe method, and the earth pressure remaining following shaking is small. For the cases with larger relative displacements, since the spread-induced 246 247 component of earth pressure is relatively large, the peak earth pressures can be larger than the 248 Mononobe-Okabe value.

When the seismic performance of a pile-supported bridge abutment is analyzed with the conventional Japanese design methods, the response coinciding with the arrival of the largest (principal) shock is generally assumed to represent the critical design condition for the abutment. However, our observations have shown that the horizontal displacement of the abutment coinciding with the principal shock is not necessarily the maximum displacement the abutment will experience. These observations underscore the importance of taking into account the permanent deformations of the surrounding ground.

256

257 *3.5. Bending moment of piles*

258 Bending moment diagrams for the piles, corresponding to the time when the abutments 259 experience maximum earth pressure, are plotted in Fig. 10 (for P-L, P-H, PH(Fix) & P-H(EQ1)). Envelopes of the maximum and minimum bending moment are also shown, with dashed lines, in 260 261 the figure, along with the bending moment at the end of the shaking. These diagrams show the 262 average moment of the pile group, because there is very little variation between the moments in all 263 the piles in the group. In every case, the large bending moment occurs at the pile head (z=0m) and 264 at the interface between the liquefiable and non-liquefiable layers (z=10m). When the abutment 265 top is constrained against horizontal displacement (P-H(Fix)), the maximum and minimum bending

266 moments are very small compared to the moments in the unconstrained abutments.

267 For the cases subjected to the inland earthquake motion, the maximum earth pressure acting on the back face of the abutment occurs at t=10.88 sec (cf. Fig. 9), and the inertia force of the abutment 268 269 reaches its maximum value slightly before this time. For the case with the subduction earthquake 270 motion, P-H(EQ1), the maximum earth pressure during the principal shock appears at t=27.52 sec, 271 which almost coincides with the maximum inertia force of the abutment, while the maximum 272 throughout the shaking is at t=37.60 sec. This is probably due to semi-long-period components of 273 input motion that follows the principal shock (a wave undulation with a period of about 2~3sec can 274 be seen following the principal shock in Fig. 5(b)). The fluctuation in the earth pressure induced 275 by this semi-long-period component of input motion, along with the forces that have already been 276 induced by significant lateral spreading, may have put the piles into a critical stress condition for 277 the case with the subduction earthquake motion.

For all the cases, when the earth pressure acting on the abutment is maximum, the bending moment at z=0m and z=10m is approximately equal to the maximum and minimum bending moments that occur throughout the shaking. For cases in which the maximum earth pressure acting on the abutment back face occurs at about the same time that the maximum inertia force occurs, (i.e., the cases with the inland earthquake motion), the critical bending moment in the piles coincides with the maximum inertia force; however, this is not true for the subduction earthquake case.

These results suggest that for the inland earthquake, critical pile bending moments can be determined by the pushover analysis (even though maximum abutment displacements cannot be determined by conventional methods). But when the critical bending moment in the piles is caused by lateral spreading that occurs after the principal shock (such as in the subduction earthquake with a relatively small peak motion but with a duration long enough to cause lateral spreading), neither the maximum pile bending moments nor the maximum abutment displacements can be assessed with the pushover analysis.

292

293 4. Forces acting on the abutment

294

The numerical analyses reveal that when the earth pressure acting on the back face (not the inertia force) of the abutment is maximum, the piles are experiencing their critical strength/ductility demand, even though the horizontal displacement of the abutment may not have reached its maximum value yet. Although the maximum earth pressure occurs at the same time as the critical bending moment in the piles, this does not necessarily mean that the earth pressure acting on the back face of the abutment is the cause of the critical bending moment in the piles. In this section,
other components of the forces acting on the abutment are examined, in order to identify relevant
forces dominating bridge abutment response for conventional seismic design methods.

303 With regard to spreading soils, the abutment acts as a restraint against lateral spread; in other words, the forces acting on the abutment are the result of interaction between the spreading soils 304 305 and the pile foundation. The NCHRP 472 Recommended Specifications for Seismic Design of 306 Bridges [12][13] implemented this concept by formulating a simplified procedure for calculating 307 the response of an abutment over liquefied ground, and Boulanger et al. [14] applied this procedure 308 to a physical model test. First, we will present the analysis results in a form similar to that used in 309 the NCHRP 472, and then we will analyze the contribution of each component of the force acting 310 on the abutment.

Figure 11 shows plots of the average foundation resisting forces, F_R/B , against the horizontal 311 displacement of the abutment base for P-L, P-H, P-H(EQ1), NP-L & NP-H. For the abutments 312 313 without piles, the resisting force is calculated by integrating the shear stress on the foundation base 314 caused by frictional resistance. The resisting force of the pile-supported abutments is determined by summing the shear forces at the pile heads (the frictional resistance of the foundation soil is 315 316 negligible for these cases). The average resisting force corresponds to the foundation resistance 317 per unit width of the abutment. The points at the end of shaking are indicated by triangles, and the 318 times of the maximum earth pressure acting on the abutment back face is shown by double circles.

For the cases subjected to the inland earthquake motion (P-L, P-H, NP-L & NP-H), the foundation resisting forces act to restrain the spreading soils. Therefore, the larger the foundation resisting force, the smaller the permanent abutment displacement, as shown in the figure. The chain lines in the figure represent the abutment displacement curves subjected to various restraining forces, and the triangles correspond to points where the displacement of the pile foundation and the abutment are compatible.

325 The foundation resistance coinciding with the maximum earth pressure acting on the back face of the abutment (the double circle points) does not seem to depend on the geometry of the 326 327 surrounding ground, but instead depends on the foundation type. However, the backbone curve 328 depends on both ground geometry and foundation type, for the cases subjected to the inland 329 earthquake motion. The slope of the backbone curves decreases with shaking, that is, it decreases 330 with the relative displacement. The average shear forces are plotted against the horizontal pile displacement at a depth of 8.75m (just above the interface between the liquefiable and 331 non-liquefiable layers) for P-L, P-H & P-H(EQ1), shown in Fig. 12. These figures indicate that 332 333 the decrease in the slope of the backbone curve that takes place as the shaking progresses may be

caused by softening of the soils in both the liquefiable layer and in the bottom non-liquefiable layer,
due to the cyclic shearing of the soil surrounding the piles. This cyclic softening effect in a soil
layer lying beneath a liquefiable layer is not ordinarily considered in pushover analyses, which
might, therefore, underestimate actual abutment displacements.

338 Comparisons between Figs. 11 and 12 bring to light the following points: (1) large changes in the shear force at the pile head during the first shock are not seen in the vicinity of the interface 339 340 between the liquefiable and non-liquefiable layers, for the cases with the inland earthquake motion, 341 and (2) after the principal shock, the shear forces near that interface are slightly larger than the shear 342 forces at the pile head. The first observation indicates that the dynamic load delivered by the pile heads during the first shock is resisted primarily by the liquefiable layer, and is not transmitted to 343 344 the bottom non-liquefiable layer. Figure 13 shows the stress paths of the liquefiable soil element 345 adjacent to the piles (x=1.25m, y=3.75m & z=-6.25m) for P-L & P-H. The large resistance of the 346 liquefiable layer at that point in time may be attributed to the dilative behaviour of the soil, as 347 shown in Fig. 13. The latter observation illustrates the existence of the waterward lateral forces against the piles at the liquefiable layer, due to the lateral spread. Although this is not insignificant, 348 349 we will not discuss this phenomenon any further, because our main concern here is the forces acting 350 on the abutment.

The foundation resisting force, $F_R = F_I + (F_B - F_F) + 2F_S$ is a reaction to the forces acting on the 351 abutment (see Fig. 14) where F_B is the forces acting on the back face of the abutment, F_F on the 352 353 front face, F_S on the side faces, and F_S is the inertia force. All forces except F_I are affected not 354 only by dynamic soil-structure interaction, but also by the spreading soils. Examining the contributions of these forces to the abutment response is of value for purposes of improving the 355 356 current design method, because identification and consideration of the relevant factors dominating 357 bridge abutment response is important in the simplified design methods. This is of value not only 358 for performance assessment of the foundation itself, but also for assessing the soils surrounding the 359 abutment, since these forces are also acting to restrain the spreading soils.

360 Changes of the forces mentioned above for P-H are shown in Fig. 15. The inertia force dominates the group of forces acting on the abutment during the principal shock. The forces 361 362 acting on the back face and side faces of the abutment increase with shaking, but the forces on the 363 front face decrease. Similar plots for NP-H are shown in Fig. 16. In this case, the forces acting 364 on the side faces are negligible. Changes in the sum of these forces, and (1) changes in the shear 365 forces acting at the pile head for P-H, or (2) changes in the integrated shear stress of the foundation 366 soil for NP-H, are plotted in the bottom of the figures. Since, (1) the force acting on a face of the 367 abutment is calculated by integrating the stresses in the soil elements that contact the abutment face of interest, and since (2) the shear force in the beam element connected to the abutment base is affected by the nodal forces acting on the opposite side of the pile heads, these forces were not exactly identical, although they were of the same order of magnitude (with an error of about 20%). Therefore, the contribution of each force acting on the abutment was determined by proportion, so that the contributing forces add up to the total force.

373 Table 3 summarizes contribution ratios of the forces acting on the abutment at the time the 374 maximum earth pressure acting on the abutment back face occurs, for P-L, P-H, P-H(EQ1), NP-L & 375 NP-H. For the cases with the inland earthquake, the inertia force is the dominant force acting on 376 the abutment base. The contribution of the force acting on the abutment side face is very small for 377 the pile-supported abutments (P-L & P-H), while it is relatively large for the abutments without 378 piles (NP-L & NP-H). For the case of the subduction earthquake, P-H(EQ1), the ground has 379 already been subjected to number of shearing cycles before arrival of the large earth pressure acting 380 on the back face. This results in a smaller inertia force, and the other components contribute more 381 to the total force acting on the abutment base.

382 A similar summary for the end of shaking is shown in Table 4. For the pile-supported abutment, the contributions of the forces acting on the abutment back face and front face $(F_B - F_F)$ 383 384 are the greatest, but there are also significant contributions of force from the side faces, especially 385 when the original ground level is high (P-H & P-H(EQ1)). However, the force acting on the side 386 faces is negative (the adjacent river dike behaving as a resistance) for the cases without piles and its 387 absolute value is larger for the case with the lower original ground level (NP-L). This is related to the relative displacement between the abutment and the spreading ground at the end of shaking, i.e., 388 389 the larger the relative displacement, the larger the contribution of the force acting on the side faces 390 (cf. Fig. 8). Figure 17 is a plot of the forces acting on the side faces against the relative displacement between the abutment and the spreading ground, for all the cases. The coefficient of 391 the force acting on the side faces is defined as a ratio of F_s to $\int_0^H L(z)\rho' g dz$, where z=depth from 392 393 the abutment top, L(z)=length of abutment in the bridge axis direction at a depth of z (i.e., the average ratio of the shear stress to the initial vertical stress). The force is uniquely related to the 394 395 relative displacement, and its upper limit seems to be a coefficient of about 0.5 in this study.

These observations suggest that, (1) the inertia force dominates the force acting on the abutment base when the peak of an earthquake motion arrives before occurrence of the major spreading of the surrounding ground, (2) the greater the restraining force of the abutment foundation, i.e., the larger the relative displacement between the abutment and spreading ground, the larger the contribution of the force acting on the abutment side faces until the side drag force 401 reaches its ultimate value. Conventional design methods employing the pushover analysis for the 402 abutment response calculation could underestimate permanent abutment displacements, unless (1) 403 the softening of the soil in the layers beneath the liquefiable layer, caused by the cyclic shearing of 404 the soils surrounding piles, and (2) the force acting on the abutment side faces, are taken into 405 account. Due consideration of these factors is recommended in order to realistically assess 406 permanent abutment displacements.

- 407
- 408 5. Conclusions
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This paper presents the results of dynamic three-dimensional finite element analyses of bridge abutments adjacent to a river dike, including the effect of liquefaction of the underlying ground using earthquake motions widely used in Japan. We examined the validity of assumptions in the conventional design methods with the pushover analysis and identified the factors dominating the abutment response through the numerical analyses. Based on the analysis results and discussion, the following conclusions can be drawn:

- 416 (1) The permanent displacements of pile-supported abutments in liquefied ground are smaller 417 than the displacements of abutments without piles, as expected. Irrespective of the 418 foundation type and elevation of the ground behind the river dike, the abutment 419 displacement that occurs with the principal shock of an earthquake is not the maximum 420 displacement; the final abutment displacement following the earthquake will be larger.
- 421 (2)The piles supporting the abutment are structurally critical at the time of the maximum earth 422 pressure acting on the back face of the abutment for both the inland earthquake and the 423 subduction earthquake, even though the dynamics of these two types of earthquakes are 424 completely different. For the inland earthquake, the maximum earth pressure coincides with the peak of the principal shock of the earthquake. At that time, the piles are in a 425 426 critical condition, because the inertia force that dominates the forces acting on the abutment 427 (about 70%) is close to the maximum. For the subduction earthquake, the major lateral 428 spreading has already started before the peak earthquake motion arrives, and the forces 429 induced by soil spreading (about 80%) overshadow the maximum inertia force (about 20%). 430 A semi-long-period component of input motion following the principal shock causes large fluctuations in the earth pressure. This fluctuation, along with the forces that have already 431 432 been induced by significant lateral spreading, puts the piles into a critical stress condition when this type of earthquake is considered. 433

434 (3) The foundation resisting force is a reaction to the combined forces acting on the abutment,

435 including the forces acting on the abutment back face, on the front face, on the side faces, 436 and the inertia force. Except for the inertia force, these forces are affected by the soil spreading. The forces acting on the back face and side faces of the abutment increase as the 437 438 relative displacement between the abutment and spreading ground far from the abutment 439 increases. The contribution of the force acting on the side faces increases along with this 440 relative displacement. In this study, the force acting on the sides faces at the end of 441 shaking is around 30-40% for pile-supported abutments, whereas this same force is 442 negligible or negative (acting as resistance for the abutment) without piles.

- (4) Conventional design methods employing the pushover analysis could underestimate the
 permanent abutment displacement unless (1) the softening of the soil in the layers beneath
 the liquefiable layer, due to the cyclic shearing of the soil surrounding the piles, and (2) the
 force acting on the abutment side faces, are taken into account. Inclusion of these factors is
 recommended for assessing permanent abutment displacements.
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- 482 306-318.
- 483
- 484 Table 1: Material parameters
- (a) For soil layers 485

Parameter	Non-li	quefiable layer	Liquefiable layer	Surface	laver	
	(Dense	e sand deposit)	(Loose sand depos	it)		
G_s		2.65	2.68	2.6	58	
e_0		0.65	0.85	0.8	35	
К		0.00054	0.00090	0.00	090	
λ		0.0072	0.019	0.0	19	
V		0.333	0.333	0.3	33	
ϕ		41.3°	38°	38	0	
ϕ_{d}		27°	27°	27	0	
μ		0.5	1.0	0.	0	
ϕ_b		19.5°	19.5°	19.	5°	
b_r		2000	400	40	0	
u_1		2	2	2		
m_1		1	1	1		
С		10	30	30)	
K_0		0.5	0.5	0.	5	
S_{ij0}		$0.7\sigma_{ij0}$	$0.2\sigma_{ij0}$	0.20	σ_{ij0}	
OCR		16	2	2		
<i>k</i> (m/s)		5×10 ⁻⁴	5×10 ⁻⁴	_	-	
(b) For abutment						
Parameter	ρ (Mg/m ³)	$EI(GN.m^2)$	EA (GN)	E(GPa)	V	
Pile	2.5	4.2	79	_	_	
Abutment	2.5	_	_	21	03	

488 489

490 Table 2: Analysis conditions

Case	Abutment	Ground level	
		from riverbed*	
P-L	w/ piles	5m	
P-H	w/ piles	10m	
P-H(Fix)	w/ piles	10m	Horizontal displacement of abutment top is constrained.
P-H(EQ1)	w/ piles	10m	Spectrum I earthquake motion (Fig. 5(b)) is applied.
NP-L	w/o piles	5m	
NP-H	w/o piles	10m	

491 * Geometry of surrounding ground for 5m is illustrated in Fig. 2(a) and that for 10m is in Fig.2(b).

492

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Table 3: Contribution ratios of the forces acting on the abutment at time of maximum earth pressureacting on abutment back face

Case	P-L	P-H	NP-L	NP-H	P-H(EQ1)		
Time: sec	10.88			27.52^{*1}	37.60^{*2}		
$(F_B - F_F)/F_R$	25%	24%	19%	25%	54%	57%	
$2F_S/F_R$	3%	6%	8%	12%	25%	36%	
F_I/F_R	73%	70%	73%	64%	21%	7%	

495 *1: At time of maximal earth pressure during principal shock

496 *2: At time of maximum earth pressure throughout shaking

498 Table 4: Contribution ratios of the forces acting on the abutment at end of shaking

Case	P-L	P-H	NP-L	NP-H	P-H(EQ1)
$(F_B - F_F)/F_R$	72%	59%	142%	105%	61%
$2F_S/F_R$	28%	41%	-42%	-5%	39%



Fig. 1: Bird's eye view of abutment for river crossing bridge









516(c) Road level plan (P-L)(f) Road level plan (P-H)517Fig. 6: Ground deformations around piled abutment, where displacement scale is magnified by a518factor of ten for P-L (Left) and P-H (right) at *t*=30sec





Fig. 7: Excess pore water pressure ratio contours at t=20sec for P-L



524 Time: sec 525 Fig. 8: Time histories of horizontal displacement of abutment top 526



Fig. 9: Time histories of coefficient of earth pressure acting on abutment back face



530 Bending moment: MN.m 531 Fig. 10: Distributions of pile bending moment at time of maximum earth pressure acting on 532 abutment back face









Fig. 13: Stress paths of liquefiable soil element adjacent to pile (x=1.25m, y=3.75m & z=-6.25m) for P-L & P-H



Fr The set of the set

(b) Force acting on front face



(c) Force acting on side face

(a) Force acting on back face



(d) Inertia force

(e) Horizontal reaction at base

Fig. 14: Forces acting on abutment





552 553



555 Fig. 17: Forces acting on abutment side faces against relative displacement between abutment and spreading ground