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AN ANALYSIS OF DAMAGE TO HANSHIN ELEVATED EXPRESSWAY DURING 1995 KOBE EARTHQUAKE

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SUMMARY

Damage to Hanshin Expressway ’Kobe Route’ P1 to P718 is described in conjunction with the structural characteristics. Although the damage level of many piers was generally very high in the Kobe Route, it is also true that the visually-judged damage of many piers is moderate or mild. The visually-judged damage level is scattered along the route; no consistent trend can be found. On the other hand, there were many piers which have large residual inclination. The direction of the residual inclination of the piers was consistent with the dominant direction of the ground motion. It is interesting to note that the large residual inclination occurred mainly in the section from P50 to P300 (east part of Kobe Route). The correlation between the damage level and the residual inclination of the piers is not identified. In the latter part of this paper, damage to the piers from P1 to P350 is investigated in detail. Considering that large residual inclination(>0.5°) of piers is earthquake-induced severe damage, it is shown that almost all the RC single piers from P35 to P350 received severe damage consistently. The ratio, *r* of flexure to shear capacity of the RC single piers from P1 to P350 was calculated from the design drawings ; it is found that , for severely damaged piers, the damage mode (flexure or shear) in the piers is fairly consistent with the value of *r*, either >1.0 or <1.0.

1. INTRODUCTION

Elevated expressway bridges in epicentral area were severely damaged or even some were completely collapsed during the Kobe Earthquake. The Hanshin Expressway ‘Kobe Route’ was not exceptional. The Kobe Route of total length 25km from Nishinomiya to Tsukimiyama (Fig.1, Pier No.1-718) was sujected to near-field strong ground motion and as a result large portion of the Kobe Route suffered serious damage. In Uozaki-Fukae (P126-142), mushroom-shaped RC slab bridges were collapsed and overturned over 600m length.

Although damage level of many piers was generally very high in the Kobe Route, it is also true that damage to many piers judged by visual inspection were moderate or mild. Fig.2 shows the photographs of two adjacent piers after the earthquake. Whereas (a)P48 is severely damaged, (b)P49 in only 35m west of P48 seems to be non-damaged.

It is most important to clarify the causes of damaged structures as well as the causes of undamaged structures based on the data obtained in the earthquake. This study describes the damage to Hanshin Expressway ‘Kobe Route’ in conjunction with the structural characteristics, and attempts to clarify the damage of each bridge from the mechanics point of view.

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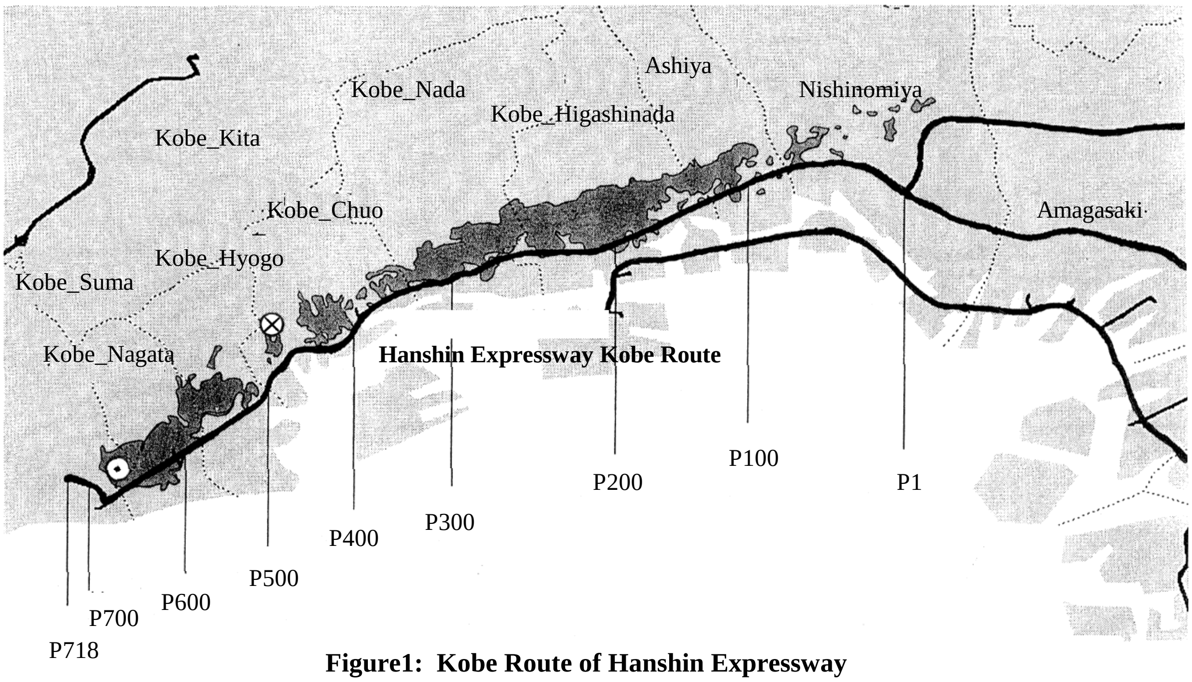


Figure1: Kobe Route of Hanshin Expressway
(black-painted part is the region where the seismic intensity is more than 7,
⊙: JR Takatori station, ⊗: Kobe marine meteorological observatory)

(a) P48 (b) P49

Figure2: Different damage level of RC single piers

2. STRUCTURE CHARCTERISTICS AND THE DAMAGE TO KOBE ROUTE

The Kobe Route was constructed from 1960's to 1970's as a part of Hanshin Expressway and almost completely adopted the elevated type. Many single piers (approx. 80%) were used to provide the space for traffic beneath the elevated expressway. The height of piers is about 10m in average and the span length along the route varies from 20m to 80m. Most of the superstructures were simply-supported spans and continuous span were employed only in long span in order to cross the major streets. Almost 90% of the girders were either I-girder or box girders. Piled foundation was extensively employed because of the ground condition.

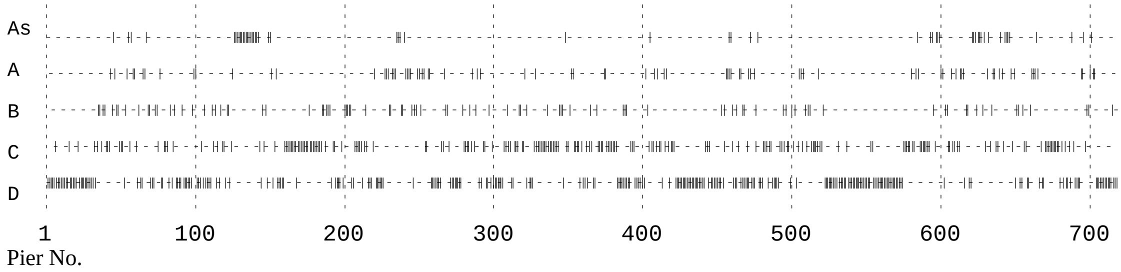


Figure4: Visually-judged damage level of the piers

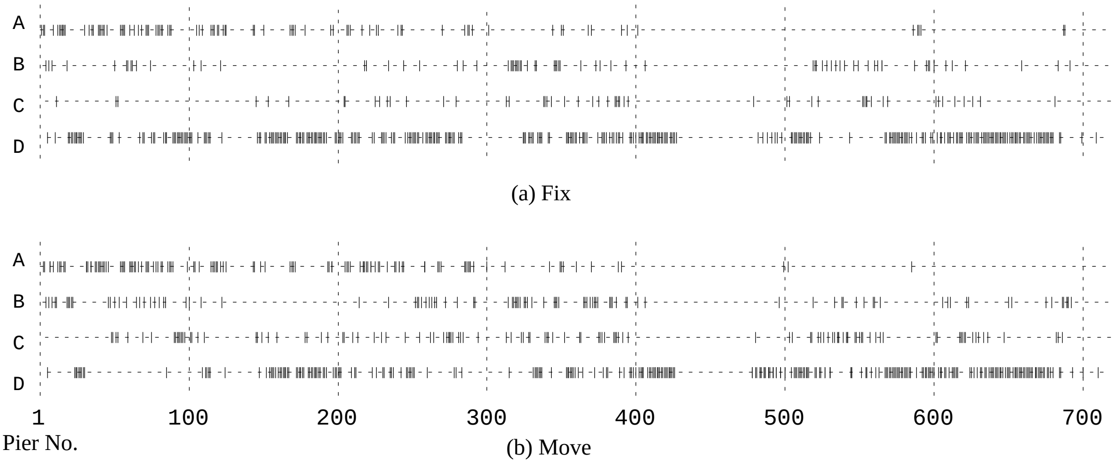


Figure5: Visually-judged damage level of the bearings

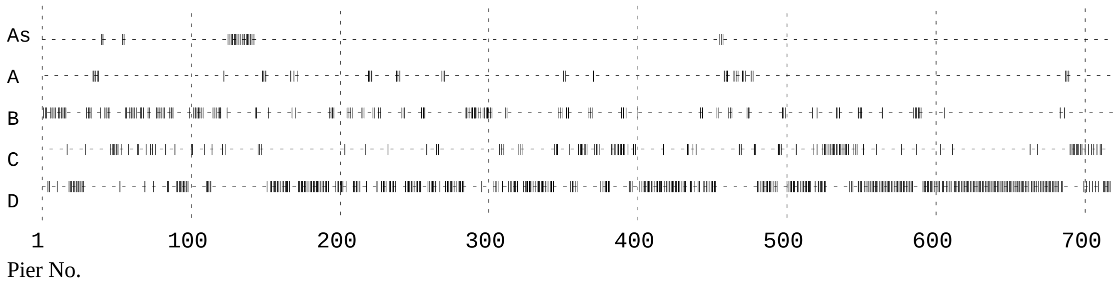
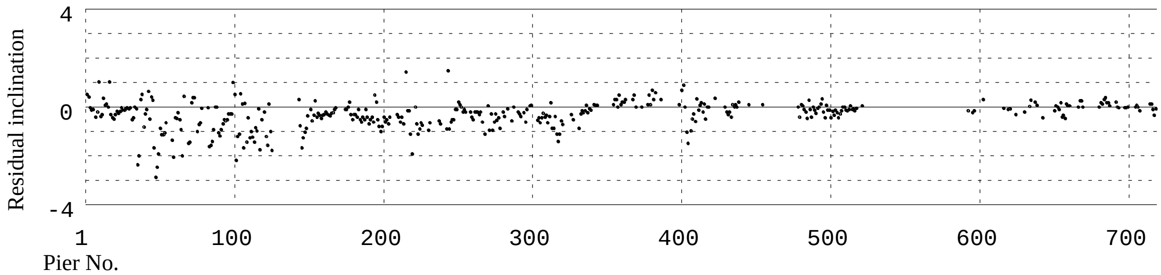


Figure6: Visually-judged damage level of the girders

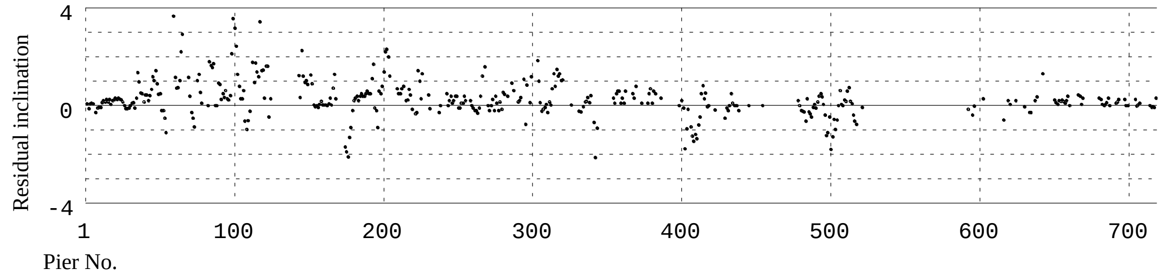
Fig.4,5 and 6 shows the damage level of the piers, bearing, girders along the route respectively. The damage level was basically based on the appearance of the above-ground damage: 'As' = collapsed, 'A' = near collapsed, 'B' = severe, 'C' = mild, and 'D' = no damage. The damage level shows that the spatial variation of the damage level is rather random along the route and no consistent trend can be found.

Right after the earthquake, reconstruction and repair of the bridge structures for restoration of the Kobe Route started. One of the measurements was the level of the inclination of the piers. Fig.7 shows the residual inclination of RC single piers. Large residual inclination of piers causes the difficulty of placing girders and visual uneasiness. Hence, piers with the inclination larger than 1° were demolished even if their visually-judged damages were mild. Indeed, 88 piers were removed simply because of large inclination.

It is interesting to note that the large inclination occurred from P50 to P300 (east part of Kobe Route). Although the piers in the west was closer to the epicenter, the inclination in the west part was smaller. Correlation between the damage level (Fig.4) and the residual inclination of the RC single piers (Fig.7) is seemingly not high.

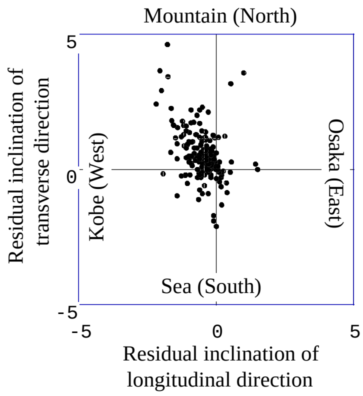


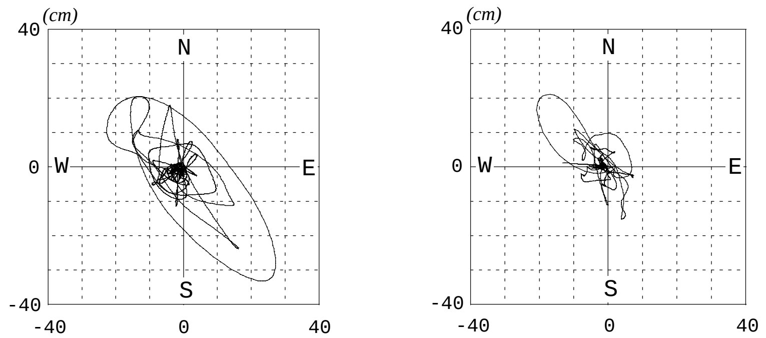
(a) longitudinal direction (＋: Osaka (East), －: Kobe (West))



(b) transverse direction (＋: Mountain (North), －: Sea (South))

Figure7: Residual inclination of RC single piers (°)





(a) JR Takatori Station (R-011)

(b) Kobe marine meteorological obs.

Figure8: Direction of Residual inclination of RC single piers (P50 – P300)

Figure9: Direction of Ground motion

Fig8 shows the direction of the residual inclination of the RC single piers from P50 to P300, while Fig.9 shows the orbit of the horizontal ground motion measured. It is clear that the dominant direction of the residual inclination is similar to the dominant direction of the ground motion.

3. DAMAGE LEVEL ANALYSIS OF SINGLE RC PIERS FROM P1 TO P350

Since the Kobe Route is very long and dealing with the whole route involves many difficult issues; primary attention is paid to the piers from P1 to P350 (25km in total) hereafter. It should be noted that almost all the piers in this section were RC singe type without cut-off of re-bars.

Fig10(a) shows the damage level of RC single piers from P1 to P350. Damage level (As, A, B, C, D) was basically based on the above-ground damage. But in the stage of restoration, the damage level was re-evaluated including the damage of under-ground piers. Bs (or Cs) means that the damage level in the underground was

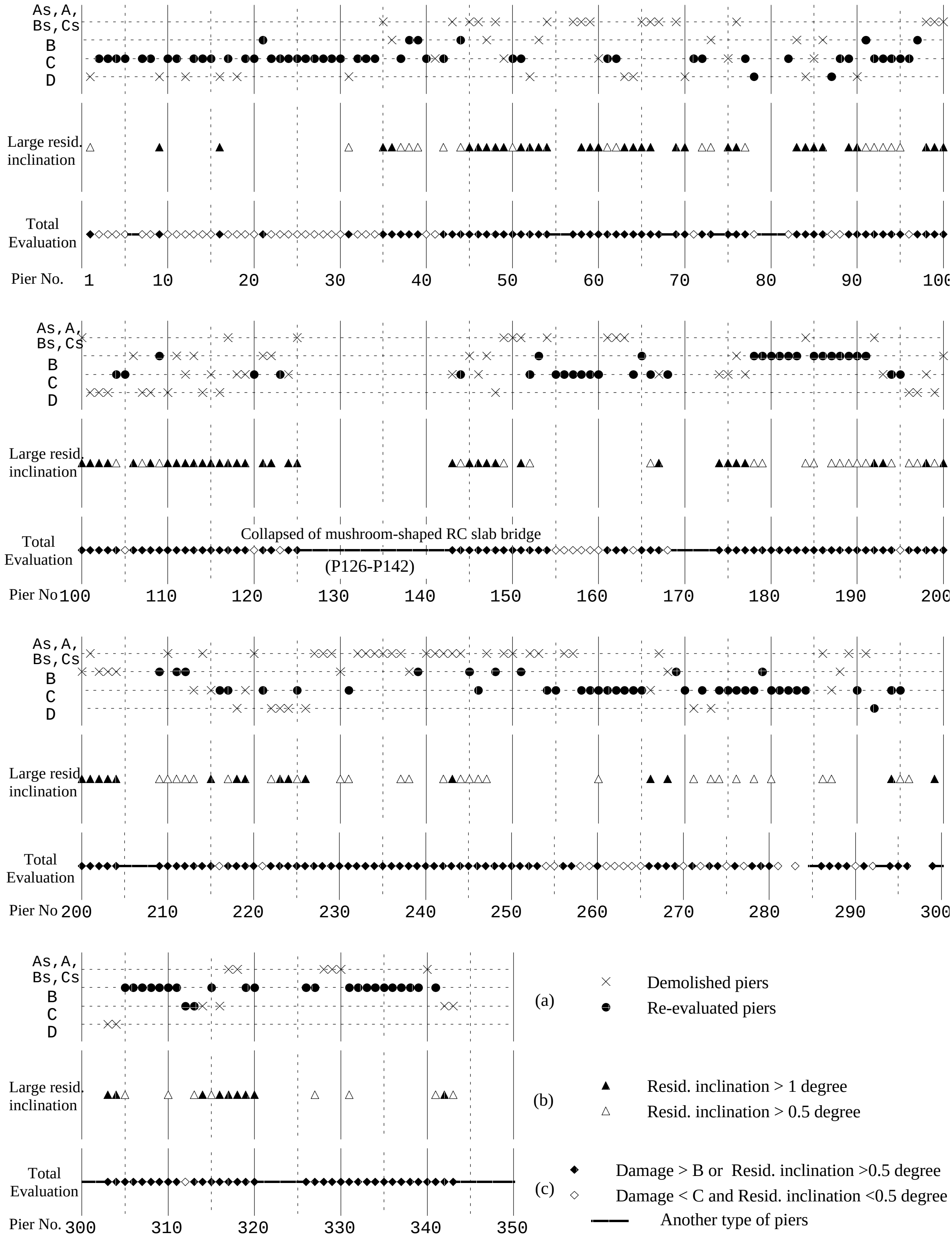


Figure10: Damage of RC single piers from P1 to P350

found to be very severe during the restoration although the damage seen above the ground level was judged to B (or C). × signs indicate the demolished piers. × sign in the first line (As, A, Bs, Cs) means that the piers were demolished because of the serious damage. On the other hand, × sign in B, C, D's line means that the piers were demolished because of large residual inclination. In Fig.10(a), the damage level is very much scattered and no consistent trend can be found. This fact is rather difficult to accept because the type of the bridge structures in this section was very similar to each other.

Fig10(b) shows the piers with residual inclination larger than 0.5°. Comparing Fig.10(a) and (b), it is clear that large residual inclination occurred in many piers without serious appearance damage.

According to Ref.1, the inclination angle 0.5° corresponds roughly to the ductility factor 2.5, and ductility factor of maximum response is estimated to be 5.0. This implies that the piers with residual inclination >0.5° suffered the damage al least B. Hence, the overall damage is defined to be either (As, A, Bs, Cs, B) or the residual inclination > 0.5° here. In Fig10(c), the piers of damage level As, A, Bs, Cs, B and with the residual inclination > 0.5° plotted as ◆ signs, and others, ◇ signs. It is clear that in the section from P1 to P34, almost all the pier are plotted as ◇ sign (light damage), and in the section from P35 to P350 , ◆ sign (severe damage). This result implies that the earthquake level would have changed around the P35.

However, there are some exceptions (◆ signs in the section from P1 to P34, and ◇ signs in the section from P35 to P350). Detailed investigation about these exceptions are carried out in Ref.1, and some of them are estimated to caused by the break of bearings (Ref.7).

4. DAMAGE MODE ANALYSIS OF RC SINGLE PIERS FROM P1 TO P350

As well known, the damage mode of RC single piers under horizontal seismic force can be classified roughly into three: either flexure, shear or their combination. Among these, the shear mode failure is non-ductile and to be avoided. Fig.11 shows the observed damage mode of RC single piers from P1 to P350. In many piers, the damage mode was flexure. Shear failure occurred, however, in some piers, which are rectangular section.

Generally the rectangular piers carried long span girders in Hanshin expressway. Fig.12 shows the typical case of reinforcing in circular and rectangular cross sections. In rectangular cross section, longitudinal reinforcements was increased to meet large bending moment due to large weight of the long span whereas the transverse reinforcement was not proportionally increased. The seismic design code employed at 1960's overestimated the shear capacity carried by concrete, and did not require adequate transverse reinforcement for shear seismic design of the piers, resulting in low shear capacity.

The flexure and shear capacity of the RC single piers from P1 to P350 are calculated from the design drawings collected from Hanshin Public Expressway Corporation (HPEC). According to the strength test (Ref.3), the strength of materials are estimated to be 38Mpa (concrete), 352Mpa (longitudinal bars), 355Mpa (transverse bars) respectively. Flexure capacity is calculated according to the design code of Japan (Ref.6). Shear capacity carried by concrete is estimated according to Ref.5 and the effect of the transverse reinforcement and the dead load is added according to the design code (Ref.2).

The ratio of shear to flexural capacity of RC piers under the horizontal load is defined as
$r \equiv v_y / v_{mu}$
v_{mu}: shear force of the cross section when its bending force reaches to the flexural capacity
v_y : shear capacity

This ratio r indicates
$r > 1$: flexure failure mode
$r < 1$: shear failure mode

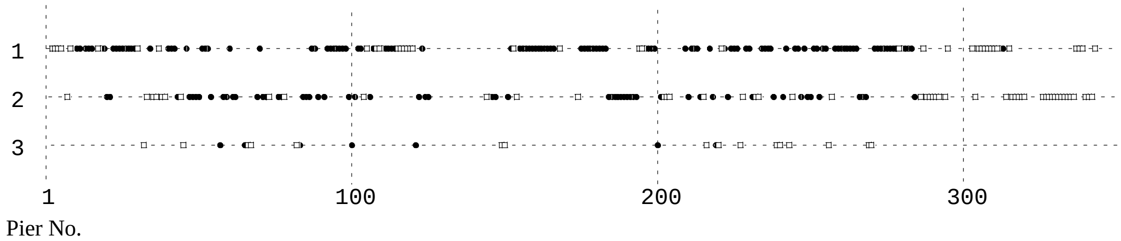


Figure11: Visually-judged damage mode of RC single piers

(1: flexure, 2: flexure - shear, 3: shear) (shapes of cross sections ●: circlular, □: rectangular)

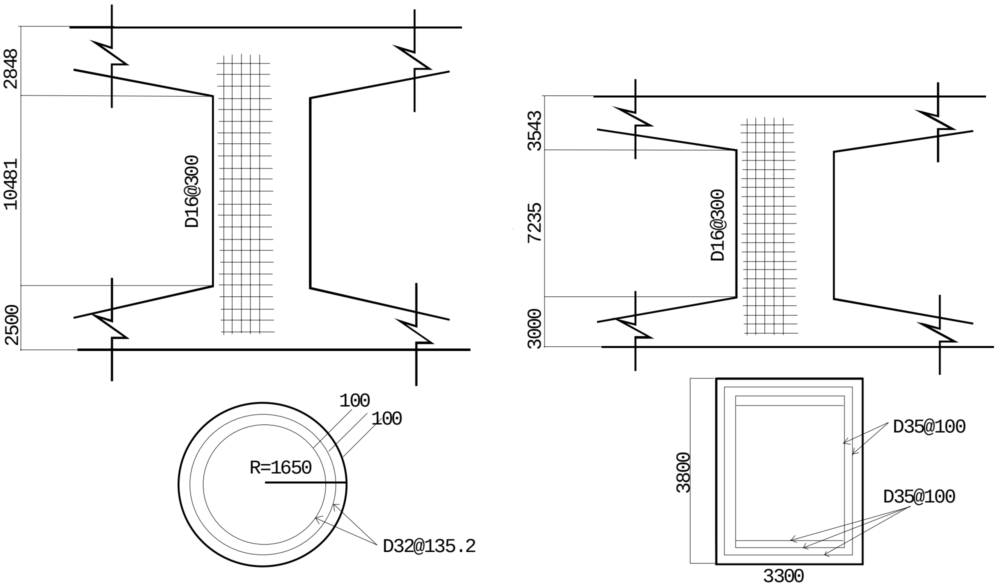


Figure12: Arrangement of reinforcing steel in typical cross sections

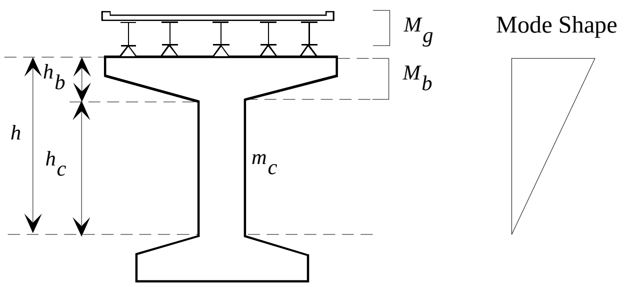


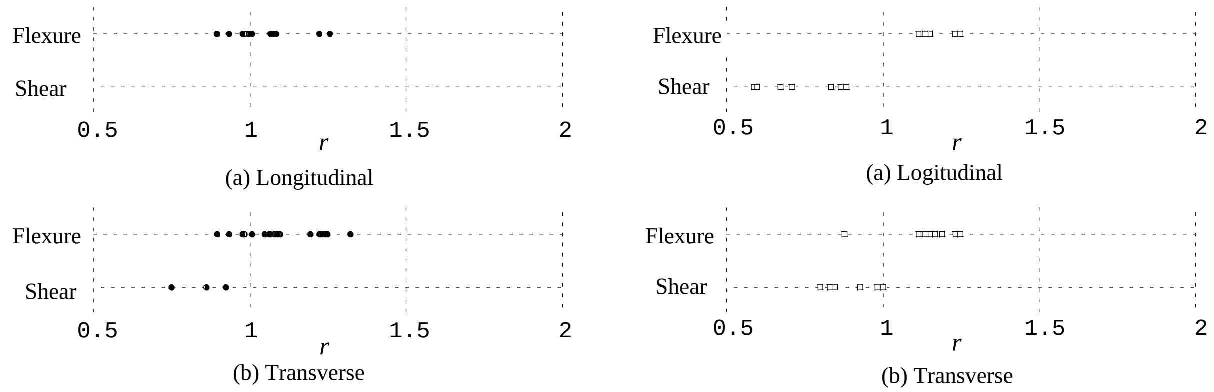
Figure13: Distribution of the mass of the bridge and the assumed mode shape

Although, v_{mu} can be approximated to be M_u/h (h: height of the pier), the effect of the mass of the pier column is considered herein. If mass of each component is assumed as shown in Fig.13, r is expresses as follows.

$$r = \frac{V_y}{V_{mu}} = \frac{M_g h + M_b(h - \frac{1}{3}h_b) + \frac{1}{2}m_c h_c^2}{M_g + M_b + m_c h_c} \times \frac{V_y}{M_u - M_e}$$

M_e: moment cased by eccentricity of the mass

Fig.14 shows the relationship between r and the observed failure mode. This plot is made only for severely damaged piers (damage level > B). In all the piers of r >1.0 occurred flexure type failure, and in almost all the piers of r <1.0 occurred shear type failure. Only in 8 piers, the failure mode was flexure in spite of the value of r <1.0. These results indicate that the prediction of failure mode according to flexure and shear capacity is in good agreement with the actual mode.



(i) Circular cross section (ii) Rectangular cross section

Fig.14 Damage mode and the ratio, r of flexure to shear capacity in severely damaged (>B) piers

5. CONCLUSIONS

Analysis on the damage to RC bridge piers of Hanshin Expressway suffered during the 1995 Kobe Earthquake was made. The study can be summarized as follows

1. The visually-judged damage level of piers were scattered along the route and no consistent trend can be found.
2. The large residual inclination in many piers was observed.
3. The damage level in piers can be explained in a consistent manner, considering both the above-ground damage and the residual inclination of piers together.
4. The ratio of shear to flexural capacity can predict the actual failure mode of RC single piers.

6. ACKNOWLEDGMENT

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