FINITE ELEMENT MODELLING
OF THE HANSHIN VIADUCT'S FAILURE IN KOBE

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ABSTRACT

The paper presents the numerical simulation of the Hanshin pier failure using a fibre beam element with shear modelling. The analyses seem to indicate that a shear failure of the piers has caused the collapse of the viaduct due to the insufficient transverse reinforcement and poor anchorage detailing of it. with respect to available empirical rules for the shear strength evaluation, the proposed model has the advantage of describing the evolution of the various damage mechanisms in r.c. element down to the final structural collapse. The insight gained on the behaviour of these structures is hopefully a first step in view of a future revision of present seismic shear design recommendations which are based on strength criteria only, while energy-based provision should clearly also be introduced.

1. THE NUMERICAL ANALYSES

The analyses presented in the following intend to focus on two construction details of the Hanshin concrete piers which played a determinant role in failure of the viaduct. The first one is the much discussed interruption of the inner rebar layer at 2.5 metres above the foundation plinth. this interruption has immediately come to the research attention since clearly the failure has localised in that section. Not enough attention has been given instead to the obvious consequence that, with the interruption of the inner rebar layer, also the inner hoop layer terminated at the same section.

This interruption has been even more critical as all hoops lacked of proper anchorage in the concrete core and therefore failed with the spalling of the outer concrete layers. The inner layer, although doubly spaced compared to the outer one (the same as for the corresponding longitudinal rebar layer, the two represented 20% of the reinforcing steel), had obviously a better anchorage given its inner position into the section.

The lack of anchorage of the transverse steel, which is the second detail to be analysed, has been included among the causes of failure but no quantitative assessment of its role has been carried out. Using current empirical formulae [1][2] for the calculation of the pier shear strength we can only choose between including this transverse steel or not. If we do include it, the shear strength of the pier comes out to be in excess of the bending one, while if, on the contrary, we do not include it, we should have had immediate shear failure even without yielding of the longitudinal rebars.

The analyses presented in the following, based on a more physical modelling of the behaviour of elements subjected to high shear, will try to provide an insight on the actual evolution of damage which may have taken place in the Hanshin piers.

Four configurations have been analysed at first by combining the two details discussed above:

• CASE A - Pier with rebars and hoop interruption and hoop cut-off strain at 2%.
• CASE B - Pier with rebars and hoop interruption and no hoop cut-off strain
• CASE C - Pier with no rebars and hoop interruption and hoop cut-off strain at 2%.
• CASE D - Pier with no rebars and hoop interruption and no hoop cut-off strain

The hoop cut-off strain is an approximation for the complete loss of anchorage which we may assume to take place after concrete spalls, which for the Hanshin piers could be estimated to occur at strains in the order of 1%. For properly anchored hoops we may assume instead that their cut-off strain does coincide with the steel max. elongation although this may be a bit optimistic.

2. THE FINITE ELEMENT MODEL

A single pier has been modelled using the finite element program FIBER with four 2D fibre beam element with shear modelling developed by the authors [3][4][5] in the last few years. A concentrated mass has been placed on top of the pier with a translational and rotational inertia equivalent to one span of the viaduct (M_t = 1100 kN, M_r = 29000 kNm^2). Full Lagrangian formulation (large displacement) has been used. The structure has been subjected to the Kaiyou Weather Bureau accelerogram with a PGA = 0.81 g.

The reinforced concrete section has been subdivided in 40 concrete fibres and 20 longitudinal steel ones. Each of the concrete fibre is laterally confined by a steel transverse fibre which models the effect of the hoops. During the analyses equilibrium is enforced between the concrete fibre and the transverse steel one. In order to do that, the proposed model while retaining most of the characteristics of a traditional fibre element, makes use of a 2D concrete constitutive model which allows for the description of the 2D stress strain field due to shear and Poisson's effect in the concrete. The following simplifying assumptions are maintained:

- Plane sections remain plane
- Shear strains are constant over the section
- Perfect bond is assumed between concrete and steel in the transverse direction

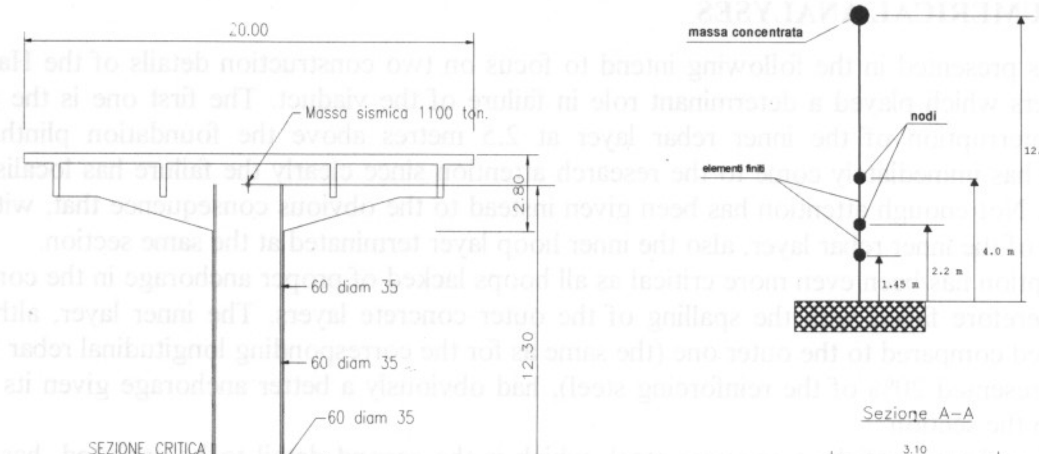


FIG. 1 - Hanshin' s pier geometry and F.E. schematisation

The steel constitutive behaviour used for the longitudinal steel is based on the Menegotto-Pinto curves [6] while for the transverse steel a simplified elasto-plastic formulation with cinematic hardening is used.

The transverse steel area attributed to the concrete core fibres is equal to the corresponding hoop area per unit length (40 to 50 cm^2). Mechanical properties of the steel are the following: yield strength = 350 MPa, ultimate strength = 450 Mpa (380 Mpa for the hoops), Young's modulus = 200000 Mpa, ultimate strain 0.12. Mechanical properties of the concrete are: cubic strength = 30 Mpa, Young's modulus = 30000 Mpa, tensile strength = 25 Mpa, fracture energy 0.1 kN/m.

Since the concrete constitutive model is a microplane based [7][8] 2D model, the base curves used for the microplane behaviour need to be defined according to the out of plane boundary conditions. In other words, the strength and ductility of concrete must be adjusted via a calibration of the base curves used by the model since the latter does not cover automatically the response of concrete due to varying stress/strain states in the third direction. In practice, the input parameters for the concrete fibres have been assigned depending on their effective confinement i.e. fiber position inside the section.

3. THE RESULTS

We shall interpret the results of the analyses with the following plots. From the pier top displacement histories plotted in Fig.2 we can see that only one structure survives, which is the pier with no rebar interruption and proper hoop anchorage (Case D). Failure of the pier with rebar interruption and proper hoop anchorage (case B) does take place later than the two cases with poor hoop anchorage (Case A and C). The shear force-pier displacement histories are plotted in the following Fig.3 (no rebar interruption) and Fig.4 (rebar interruption).

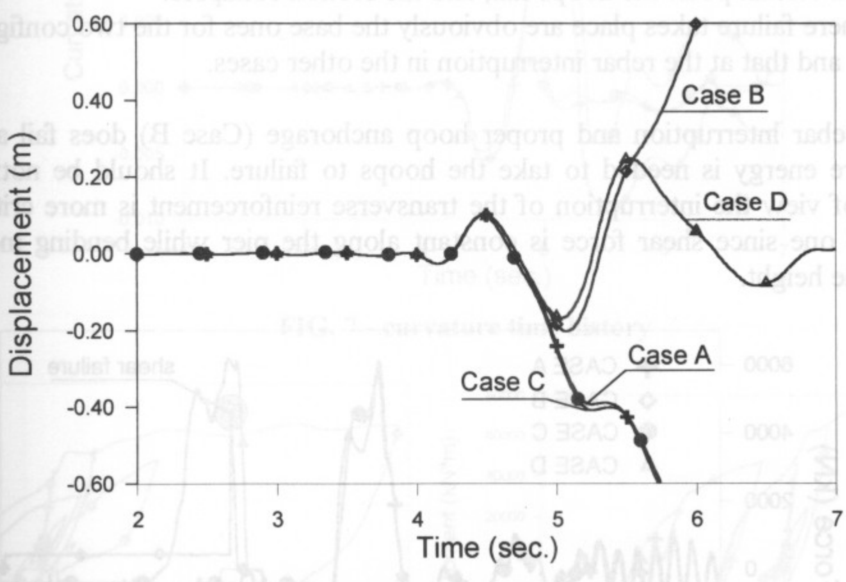


FIG. 2 - Pier Top Displacement History

It does come out immediately how the dissipated energy of the survived pier exceeds that of the other three structures. It must be remembered that the piers do not behave as simple cantilevers since there exists a mass rotational inertia of the deck; this is why Fig. 3 and 4 show sometimes increasing forces associated to decreasing displacement due to the out-of-phase response of the rotational mass. Nonetheless, the bending moments due to this rotational mass are not particularly significant and do not cause an increase of the maximum shear force compared to that which would be obtained by a cantilever type of response. This is caused by the fact that the maximum bending moments at the pier base and pier top, although not in phase, do not happen to be in opposition of phase.

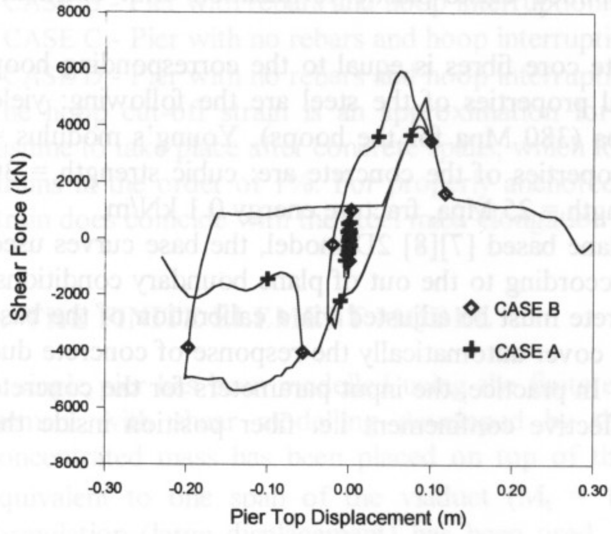


FIG. 3

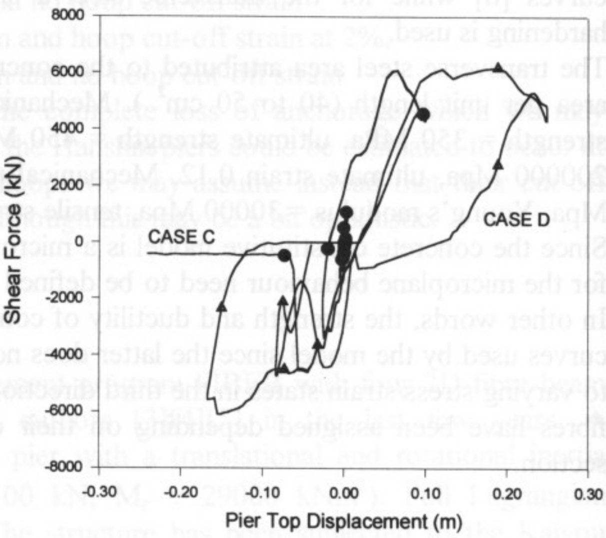


FIG. 4

The most significant plots illustrating the failure of these structures are certainly the shear force and shear deformation time-histories of Fig.5 and Fig.6 respectively. Shear failure is there, brittle as it is expected to be, following previous peak corresponding to diagonal cracking of the section. The numerical simulation shows in fact the development of a diffuse shear crack pattern which drastically reduces the shear strength and stiffness of the section and activates the transverse steel resisting mechanisms: at a certain point the hoops fail, and the section collapses.
The sections where failure takes place are obviously the base ones for the two configurations without bar interruption and that at the rebar interruption in the other cases.

The pier with rebar interruption and proper hoop anchorage (Case B) does fail as well, although later, since more energy is needed to take the hoops to failure. It should be noticed that from a strength point of view the interruption of the transverse reinforcement is more critical than that of the longitudinal one since shear force is constant along the pier while bending moment decreases linearly along the height.

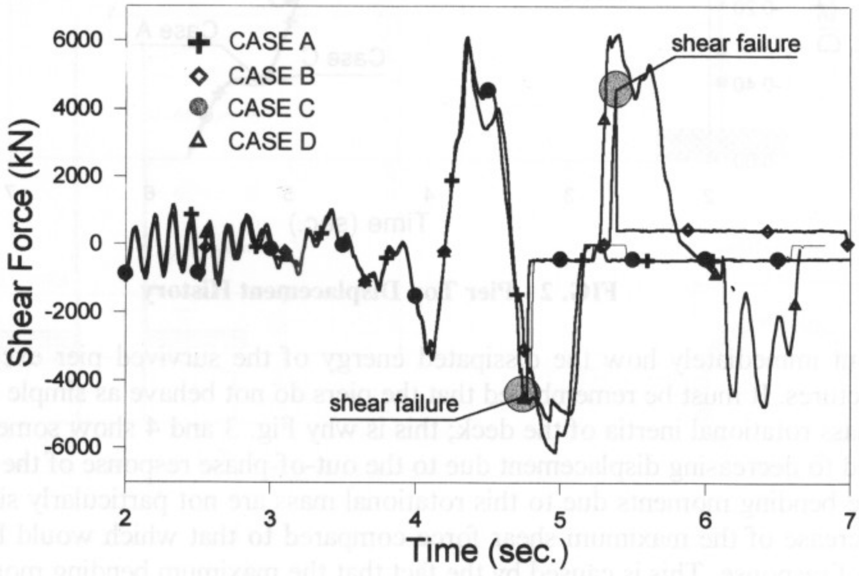


FIG. 5 - Shear force time history

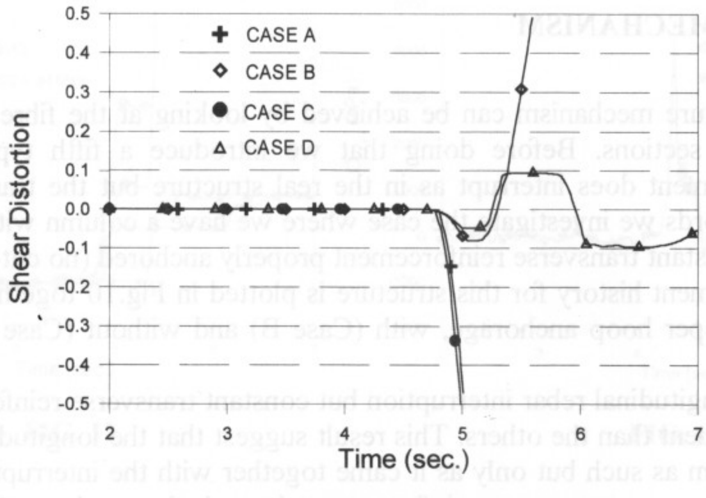


FIG. 6 - Shear deformation time history

Further evidence on the driving role played by the shear failure in the collapse of the viaduct can be seen from the curvature time histories of the collapsed section, Fig. 7 and their bending moment-curvature histories, Fig.8 and Fig.9.

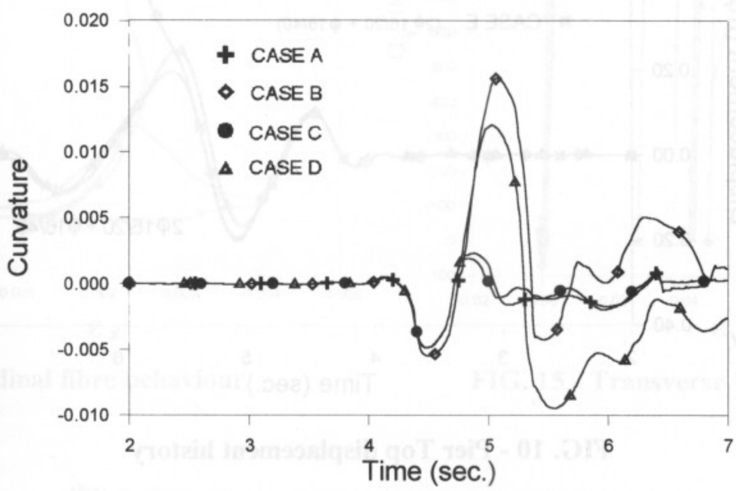


FIG. 7 - curvature time history

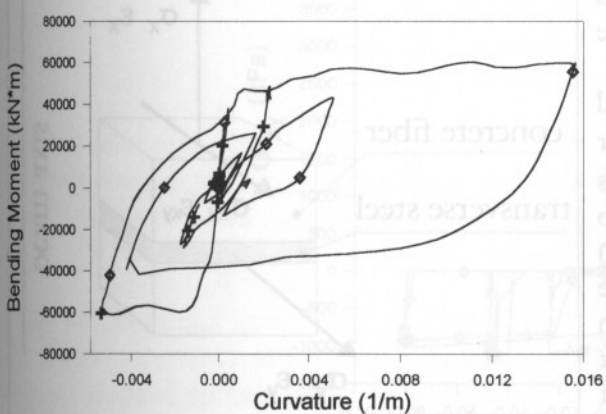


FIG. 8- Moment Curvature (h=2.5m)

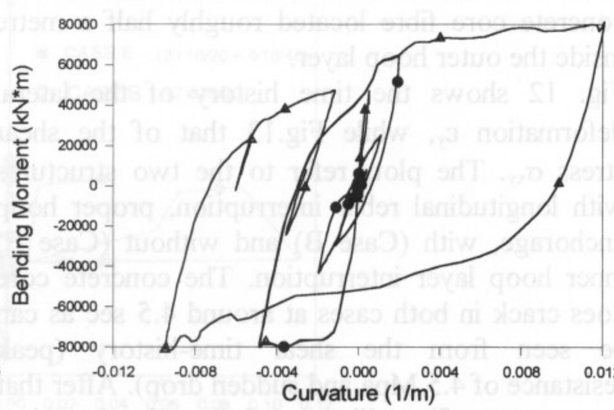


FIG. 9 - Moment-Curvature (h=0)

While the shear distortion of these sections increases dramatically, the flexural response shrinks. Only the pier with no rebar interruption and proper hoop anchorage is capable of resisting the imposed acceleration with a flexural mechanisms and a large amount of dissipated energy.

4. THE FAILURE MECHANISM

An insight on the failure mechanism can be achieved by looking at the fibre stress-strain histories inside the collapsed sections. Before doing that we introduce a fifth type of pier where the longitudinal reinforcement does interrupt as in the real structure but the transverse hoops do not (Case E). In other words we investigate the case where we have a column with varying longitudinal reinforcement but constant transverse reinforcement properly anchored (no cut-off strain).
The pier top displacement history for this structure is plotted in Fig.10 together with the two other previous cases of proper hoop anchorage, with (Case B) and without (Case D) longitudinal rebar interruption.
The structure with longitudinal rebar interruption but constant transverse reinforcement does survive with smaller displacement than the others. This result suggest that the longitudinal rebar interruption has not been a problem as such but only as it came together with the interruption of the inner hoop layer. By keeping the same transverse reinforcement through the weak section at 2.5 metres, the piers behave as well as or even better than the structure without rebar interruption.

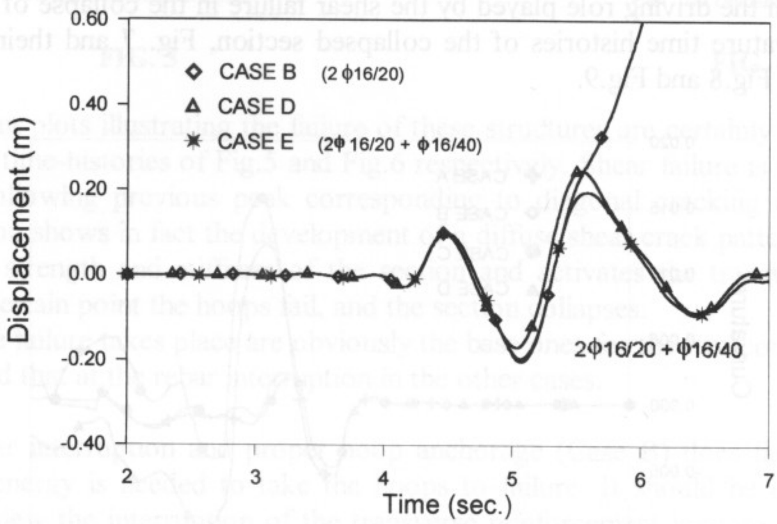


FIG. 10 - Pier Top displacement history

With reference to the notation in Fig.11, we look now at the stress and the strain time-histories of a concrete core fibre located roughly half a metre inside the outer hoop layer.
Fig. 12 shows the time history of the lateral deformation ε_y, while Fig.13 that of the shear stress σ_{xy}. The plots refer to the two structures with longitudinal rebar interruption, proper hoop anchorage, with (Case B) and without (Case E) inner hoop layer interruption. The concrete core does crack in both cases at around 4.5 sec as can be seen from the shear time-history (peak resistance of 4.5 Mpa and sudden drop). After that the concrete fibre dilates and is resisted against shear opening by the transverse steel until the latter collapses: this occurs for the case of transverse hoop interruption only. The corresponding stress-strain histories for the longitudinal, transverse and shear components for the same concrete core fibre are plotted in Fig.14 to Fig.16.

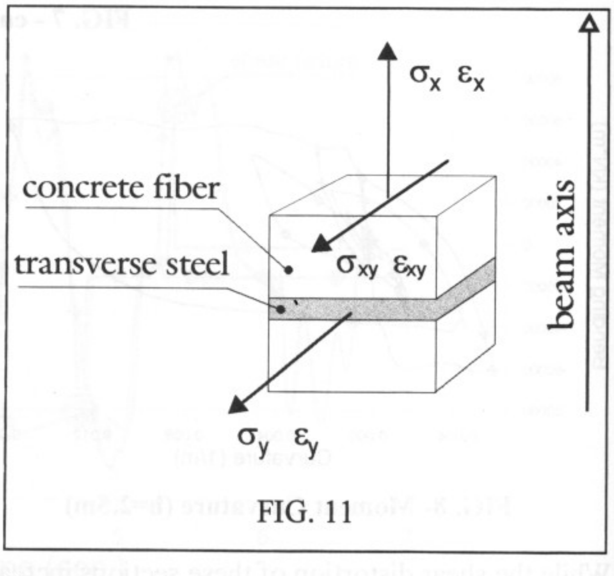
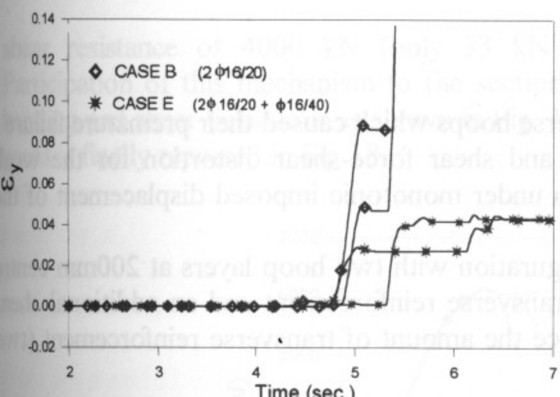


FIG. 11



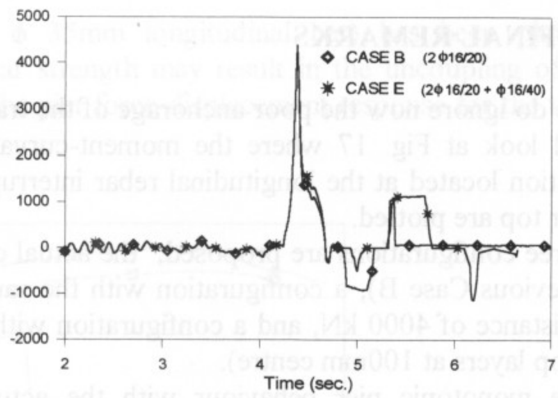


FIG. 12 **FIG. 13**

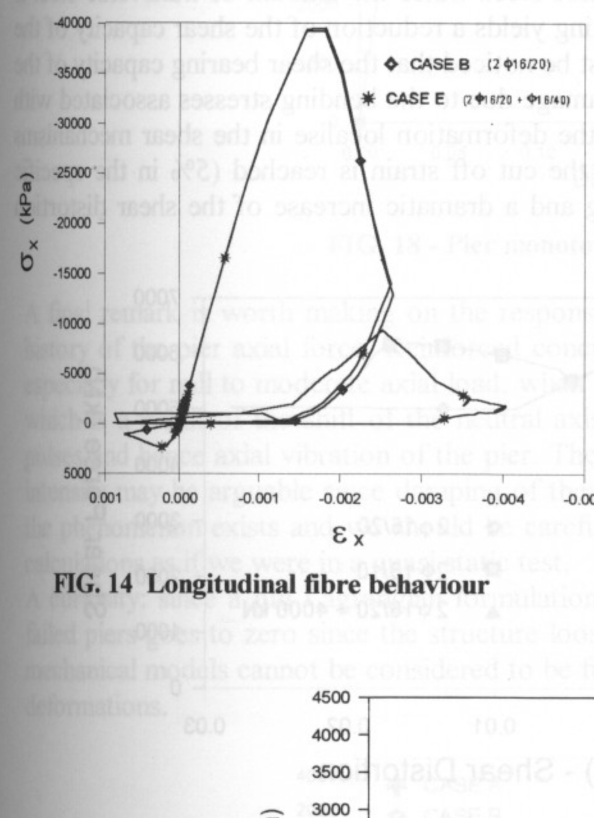


FIG. 14 Longitudinal fibre behaviour

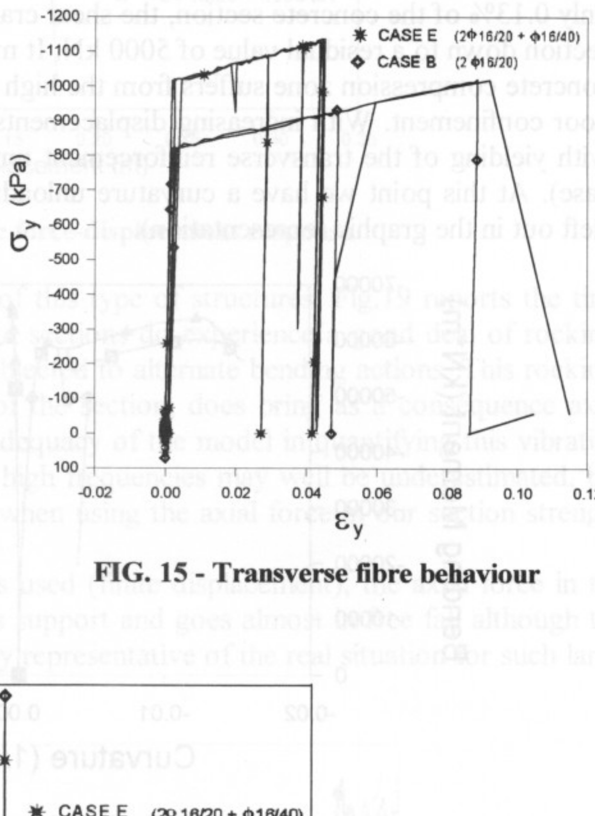


FIG. 15 - Transverse fibre behaviour

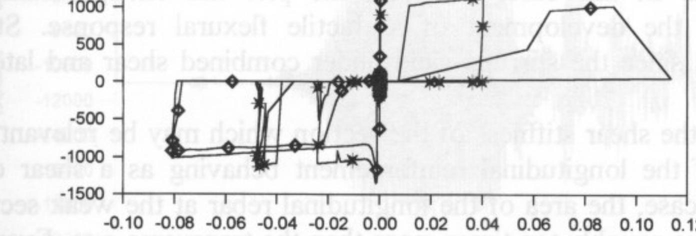


FIG. 16 - Fibre shear behaviour

5. FINAL REMARKS

We do ignore now the poor anchorage of the transverse hoops which caused their premature failure and look at Fig. 17 where the moment-curvature and shear force-shear distortion for the weak section located at the longitudinal rebar interruption under monotonic imposed displacement of the pier top are plotted.
Three configurations are proposed; the actual configuration with two hoop layers at 200mm centre (previous Case B), a configuration with the same transverse reinforcement and an additional shear resistance of 4000 kN, and a configuration with twice the amount of transverse reinforcement (two hoop layers at 100mm centre).
The monotonic pier behaviour with the actual configuration (Case B) is the following: with increasing displacement the section yields at about 60000 kN*m, shortly afterwards the concrete section develops diagonal cracks which cut through the compression zone. This crack pattern activates the truss mechanism based on the transverse steel. Since the amount of transverse steel is only 0.13% of the concrete section, the shear cracking yields a reduction of the shear capacity of the section down to a residual value of 5000 kN. It must be noticed that the shear bearing capacity of the concrete compression zone suffers from the high damage due to the bending stresses associated with poor confinement. With increasing displacements, the deformation localise in the shear mechanisms with yielding of the transverse reinforcement until the cut off strain is reached (5% in the specific case). At this point we have a curvature unloading and a dramatic increase of the shear distortion (left out in the graphic representation).

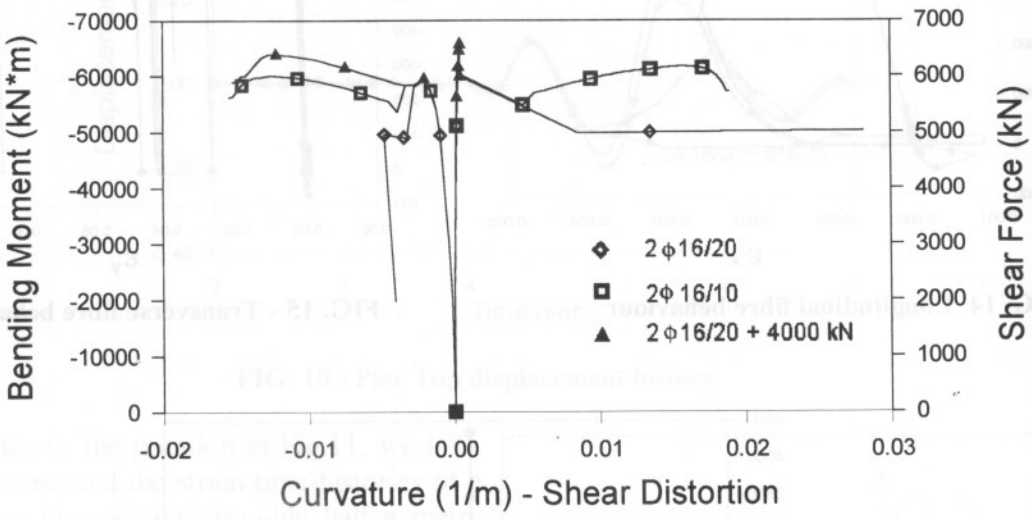


FIG. 17 - Monotonic Section Behaviour

Doubling the transverse reinforcement, the pier behaves similarly to the previous case until the truss mechanism is activated. In this case, however, the pier has enough transverse steel strength, therefore allowing for the development of a ductile flexural response. Still, significant shear deformations are found since the stirrups yield under combined shear, and lateral expansion of the compression core.
A final contribution to the shear stiffness of the section which may be relevant and should be taken into account is that of the longitudinal reinforcement behaving as a shear connector across the cracks. In the Hanshin case, the area of the longitudinal rebar at the weak section is equal to 1154 cm^2 (1.5%), which means roughly ten times more than the transverse one. Even though the strength and stiffness of this contribution decrease with increasing bending deformations of the section due to spalling of concrete and buckling of the rebars, still in case of low ductility and for sections with proper transverse reinforcement detailing, this contribution cannot be ignored. The effect of this mechanism can be seen in Fig.17 where the response of the pier section when adding an additional

shear resistance of 4000 kN (only 33 kN per ϕ 35mm longitudinal bar) has been plotted. Participation of this mechanism to the section shear strength may result in the uncoupling of the bending and shear behaviour as shown in Fig.17. The pier force-displacement response for the three cases is finally reported in Fig.18.

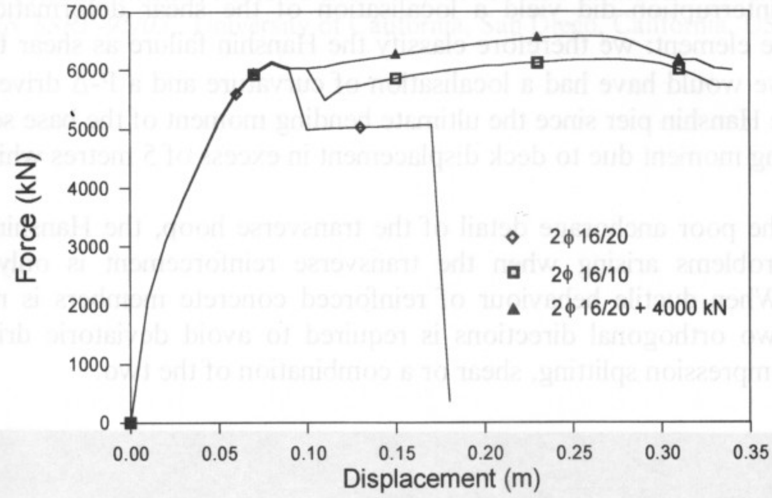


FIG. 18 - Pier monotonic force-displacement response

A final remark is worth making on the response of this type of structures. Fig.19 reports the time history of the pier axial force. Reinforced concrete sections do experience a good deal of rocking, especially for null to moderate axial load, when subjected to alternate bending actions. This rocking, which is a result of the shift of the neutral axis of the section, does bring as a consequence axial pulses and hence axial vibration of the pier. The adequacy of the model in quantifying this vibration intensity may be arguable since damping of these high frequencies may well be underestimated, but the phenomenon exists and we should be careful when using the axial force in our section strength calculations as if we were in a quasi static test.

A curiosity: since a full Lagrangian formulation is used (finite displacement), the axial force in the failed piers goes to zero since the structure looses support and goes almost in free fall although the mechanical models cannot be considered to be fully representative of the real situation for such large deformations.

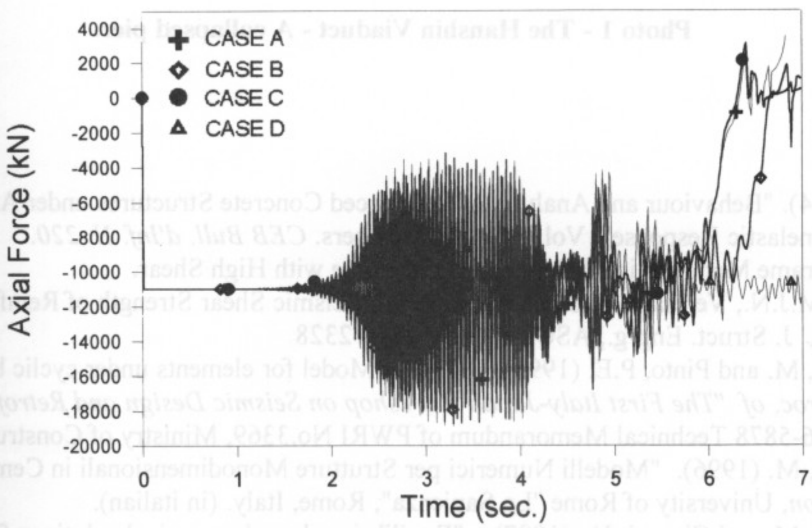


FIG. 19 - Axial Force Time History

6. CONCLUSION

The results of the analyses seem to indicate that the Hanshin's piers have failed because of lack of transverse reinforcement especially as it suddenly changed from one section to the other under an almost constant shear force (cf. [9]). Failure of the section located at the longitudinal and transverse inner rebar layer interruption did yield a localisation of the shear deformation and a curvature unloading along the element: we therefore classify the Hanshin failure as shear failure. With a more slender structure we would have had a localisation of curvature and a P-Δ driven failure which was not the case for the Hanshin pier since the ultimate bending moment of the base section could bear an overturning bending moment due to deck displacement in excess of 5 metres while failure took place at only 20 cm.

Independently of the poor anchorage detail of the transverse hoop, the Hanshin piers represent an example of the problems arising when the transverse reinforcement is only a fraction of the longitudinal one. When ductile behaviour of reinforced concrete members is required, a smeared reinforcement in two orthogonal directions is required to avoid deviatoric driven brittle failures, caused either by compression splitting, shear or a combination of the two.

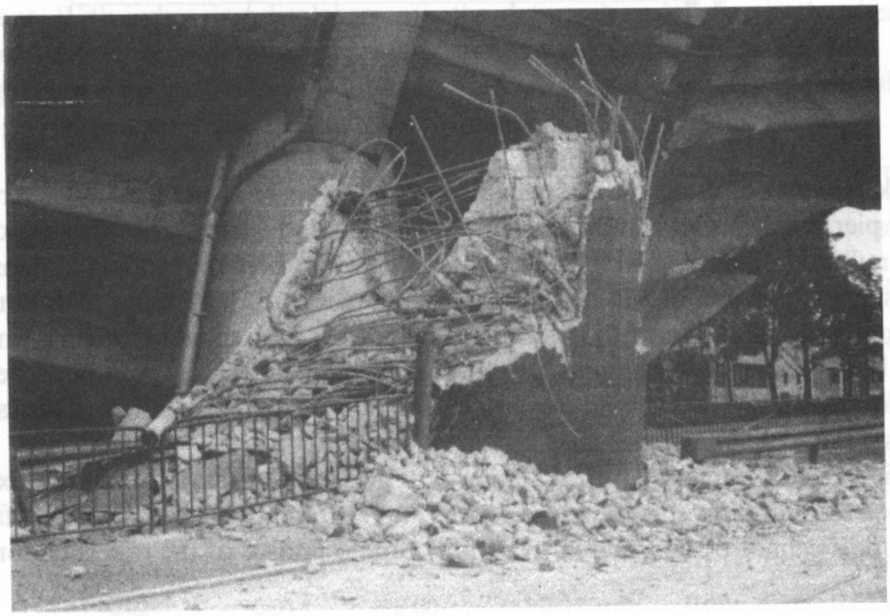


Photo 1 - The Hanshin Viaduct - A collapsed pier.

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