Lateral Deformational Behaviors of Double Box Reinforced Concrete Ducts for Emergency Cooling Water Pipes

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1. Introduction

A limit state design code for reinforced concrete structures was enforced in October 1986 by Japan Society of Civil Engineers. An attempt is now being made to apply the new code also to important civil engineering structures to be installed in nuclear power stations such as reinforced concrete ducts for emergency cooling water pipes (abbreviated as R.C.D.E. below), intake water pits, etc. It is expected the new code will not only streamline but also economize the structures. Since this type of structures must be designed against the most severe conditions of earthquake, it is of cardinal importance to investigate the mechanical behaviors of the structures during and after earthquake events.

The authors reported experimental and analytical results of a loading test of a single box R.C.D.E. at the previous SMiRT Conference(Aoyagi et al,1987). As an extension of the foregoing study this paper deals with the lateral deformational as well as cracking and strength behaviors of double box R.C.D.E. which is buried under ground. The main purposes of the study are to obtain fundamental data necessary for limit state design and to validate the non-linear finite element method developed by the authors.

2. Outline of test model and experimental procedures

A prototype double box R.C.D.E. was designed by the conventional allowable stress code on the condition that the structure was buried in relatively dense sandy soil under the earth cover of 2.9m with the level of ground water being 10m below ground surface. Design earthquake was assumed to be No.6 specified artificial earthquake with a maximum free ground acceleration of 388gal.
The outer dimensions of the R.C.D.E. were 8.8m in width and 4.7m in height with wall thickness of 0.6m.

A test model was constructed scaling down the prototype structure with a scale of 1:2. Fig-1 shows the configuration and the reinforcement arrangement of the model. Nominal shear reinforcements were also arranged.Loading scheme is illustrated in Fig-2. In the first stage of test a combination of loads was applied to the model by circumferencially distributed hydraulic jacks, simulating the conditions of earth and water pressures at the service load level. Then horizontal forces, which correspond to the load calculated by an earthquake response displacement analysis, were exerted keeping the above mentioned service load constant. The horizontal loads were introduced stepwise up to failure. Deformations as well as strains in concrete and reinforcements were measured along with the applied loads.

3.Experimental Results

Behaviors at service load level

At service load level any cracks could not be detected mainly due to the effect of compressive axial stresses induced in the members. Also at design earthquake load no cracks were visible. The allowable crack width of 0.3mm for this type of underground structure was observed only at the first yielding of reinforcement had been measured.

Horizontal deformational behaviors above design earthquake load level.

The relationships between horizontal load and deformation are depicted in Fig-3. First visible crack was observed at a load of 8.4tonf. Judging from the turning point of the curve at a load of about 6tonf, minute cracks might have precedingly occurred. First yielding in tensile reinforcement was measured near the left-upper corner of the model marked as ①. Corresponding horizontal deformation at the top of the model was defined as yield deformation δ_y . Yielding in steel around the corner regions proceeded in the order of the encircled number in Fig.3.

Ultimate behaviors

Crushing of concrete in compression zones occurred in the order of the number shown in square mark in Fig-3. The ultimate state in the static system was reached when the section $\boxed{4}$ failed. The value of the maximum horizontal load was 24.6tonf, which was equal to 4.5 times the design earthquake load, and the corresponding deformation was 63.9mm, that is approximately $21\delta_y$, which exhibited ample ductility up to failure.

4.Outline of non-linear FEM analysis

The analytical method employed was a non-linear finite element approach assuming smeared cracking. Finite element mesh adopted for analysis is drawn in Fig-4. Concrete elements were a non-linear type in which effect of cracking was incorporated. Constitutive equation for concrete obeys the law of homogeneity before cracking. Condition for cracking was determined by the failure criteria considering bi-axiality. After cracking the stiffness in the direction perpendicular to cracks were decreased based on deterioration of bond between reinforcement and concrete. Basic assumptions for FEM are shown in Appendix.

5.Comparisons between analytical and experimental results

In Fig-3 a comparison is made between analytical and experimental results of horizontal deformation at the top of the model. Fairly good correlation can be seen up to the deformation value of about 60mm, that is, $20\delta_y$. Cracking patterns also could be simulated by the analysis even up to the ultimate stage. Crushing area of concrete also could be predicted as shown in Fig-5.
From the comparisons of analytical moment distributions calculated by elastic frame analysis, elastic FEM and non-linear FEM, it was surmised that a favorable effect of moment redistribution at ultimate loading stage might be expected especially in peak moment region(see Fig-6). The maximum moment deduction rate of nearly 20% was found to be reasonably assumed for the design maximum moment.

6.Conclusions

Following conclusions were obtained from the above mentioned experimental and analytical study.

(1) From the loading test on 1:2 scale model of a double box reinforced concrete duct for emergency cooling water pipes, which was designed based on allowable stress design, it was suggested that the conventionally designed structure has an over-sufficient safety against specified earthquake loads. Application of the newly drafted limit state design has a high possibility of rendering the design of the structure not only rational but economical.
(2) Accuracy as well as reliability of the non-linear finite element method developed by the authors were confirmed in the light of comparison with experimental results. Also moment redistribution at ultimate stage could be analytically predicted.

Reference

Y.Aoyagi,T.Endo and K.Iida "Cross sectional design of reinforced concrete ducts for accommodation of emergency cooling water pipes" Transactions of 9th SMiRT Volume A pp171 176, Lausanne 1987

Appendix

Basic Assumptions for FEM

I . Failure criteria for bi-axial stress

① tension-tension field

$$\sigma_1/\sigma_2 \geqq 1.0-\alpha\,\sigma_2/\sigma_1$$

② compression-tension field(tension failure)

$$(\sigma_1/\alpha\,\sigma_t)^2+(\sigma_2/\sigma_{cu})^2 \geqq 1.0$$

③ compression-compression field

$$\sigma_2/\sigma_{cu} \geqq 1.0+a(\sigma_1/\sigma_{cu})+b(\sigma_1/\sigma_{cu})^2$$

α =0.300,a=0.831,b=0.643 (experimentally determined coefficients)
σ_t =30kgf/cm^2 (tensile strength)
σ_{cu}=300kgf/cm^2 (compressive strength)

II . Non-linear constitutive equations

① $$G_t/G_0=\frac{\{1-\alpha\,(\tau_{oct}/\tau_{cu})^m\}^2}{1+(m-1)\alpha\,(\tau_{oct}/\sigma_{cu})^m}$$

② $$K_t/K_0=\frac{G_t/G_0}{\exp(-[C\gamma'_{oct})^p\{1-p(C\gamma_{oct})^p\}])}$$

K_t:volume modulus(K_0:initial) ,
G_t:shear modulus(G_0:initial),
τ_{oct} and γ_{oct}:octahedral shear stress and strain
$\gamma'_{oct}=\gamma_{oct}/2$,
α =3.4,m=2.4,c=420,p=2.2(experimentally determined coefficient)

III . Residual stress and stiffness after cracking

① Residual stress and stiffness in the direction perpendicular to a crack are calculated by applying following residual factor

$$R=\exp(-\frac{\varepsilon_{ci}-\varepsilon_t}{\varepsilon_0})$$

ε_{ci} :average axial strain perpendicular to a crack
ε_t :axial strain at cracking, ε_0=833× 10^{-6} (experimentally determined)

② When the stress in the direction perpendicular to a crack satisfies the failure criteria, both residual stress and stiffness are reduced to zero in all directions.

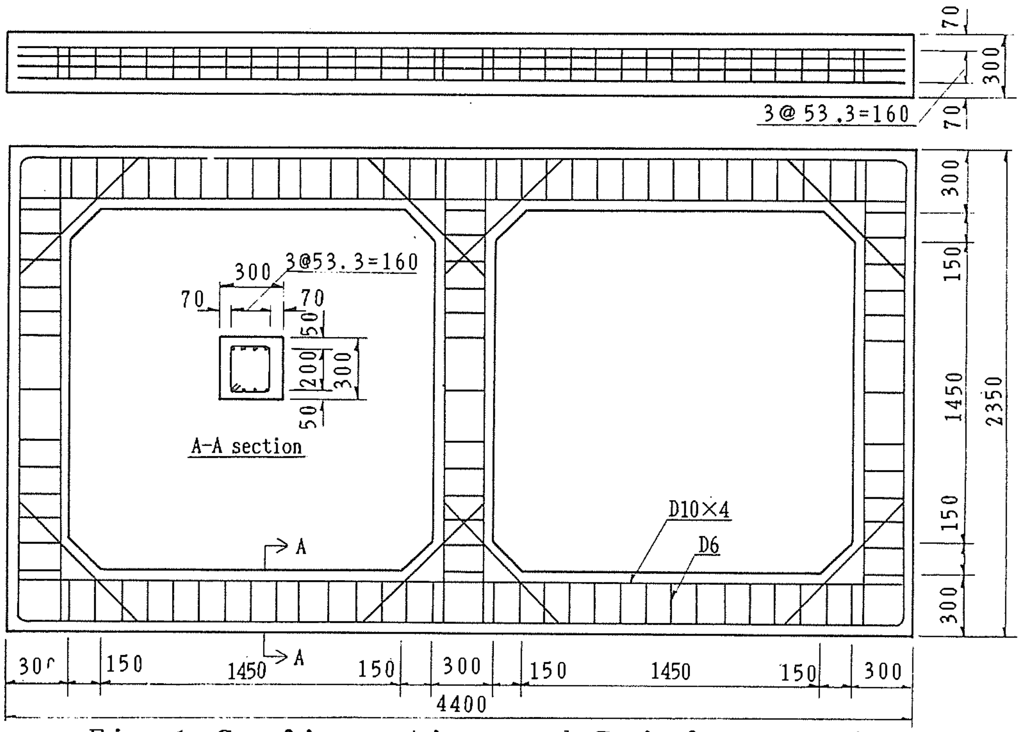

Fig-1 Configuration and Reinforcement Arrangement of 1:2 Scale Model

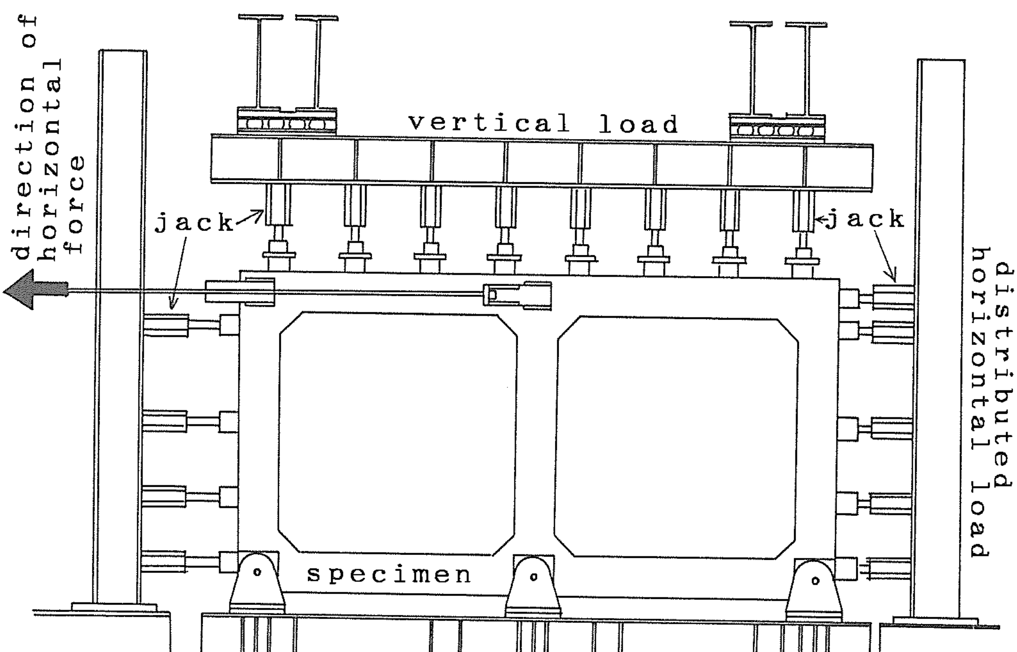

Fig-2 Loading Scheme

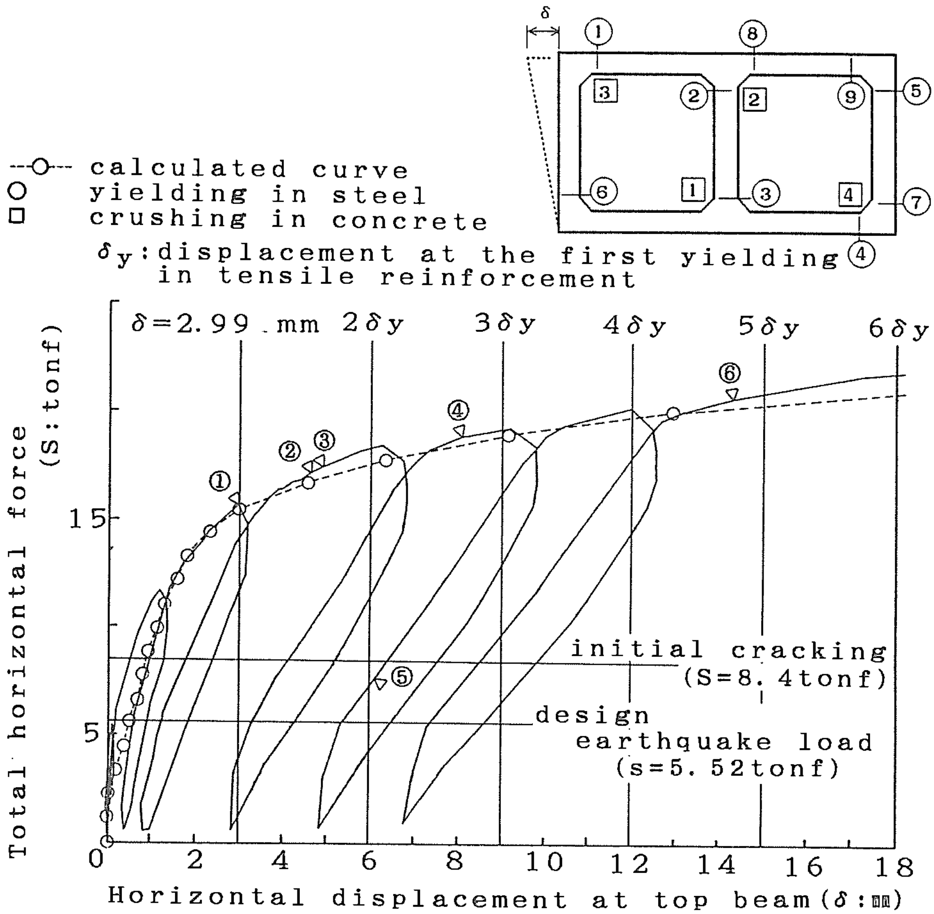

Fig-3 Relationship between Total Horizontal Load and Displacement at the Top of Model

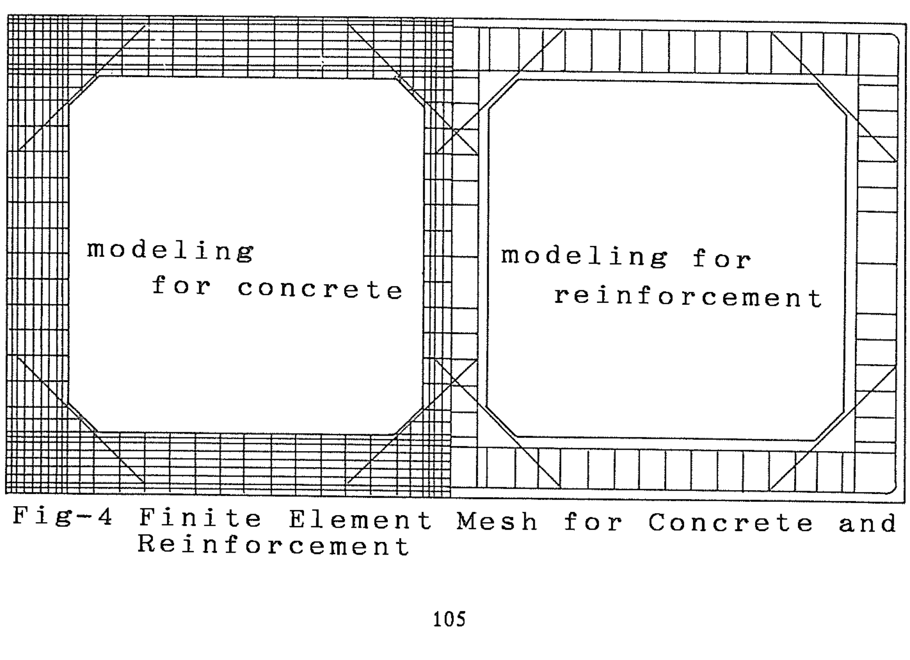

Fig-4 Finite Element Mesh for Concrete and Reinforcement

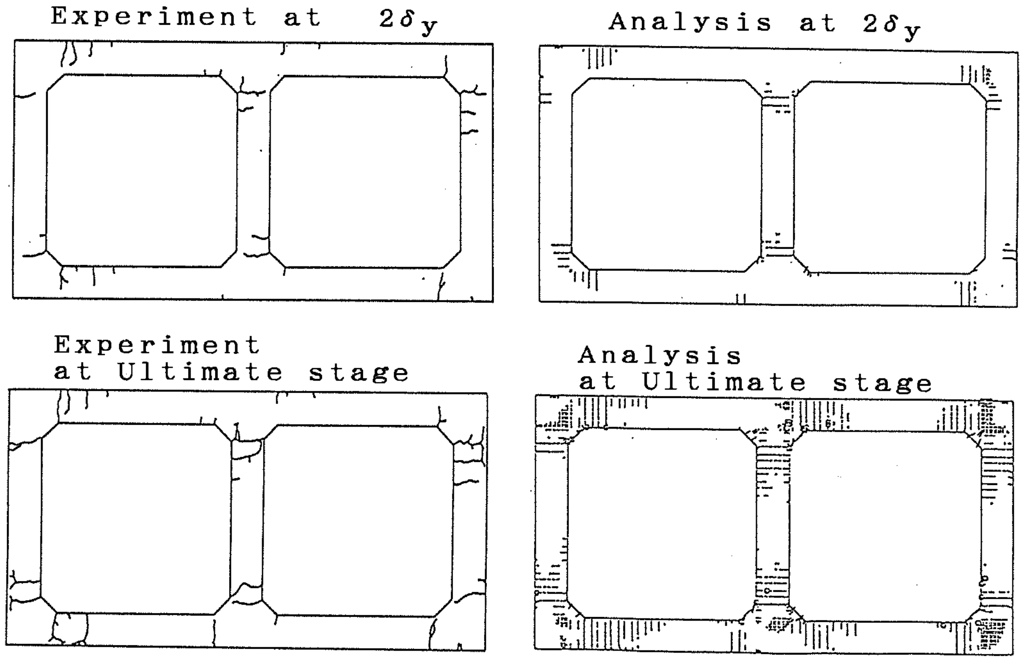

Fig-5 Comparisons of Cracking Patterns Obtain from Experiment and Analysis

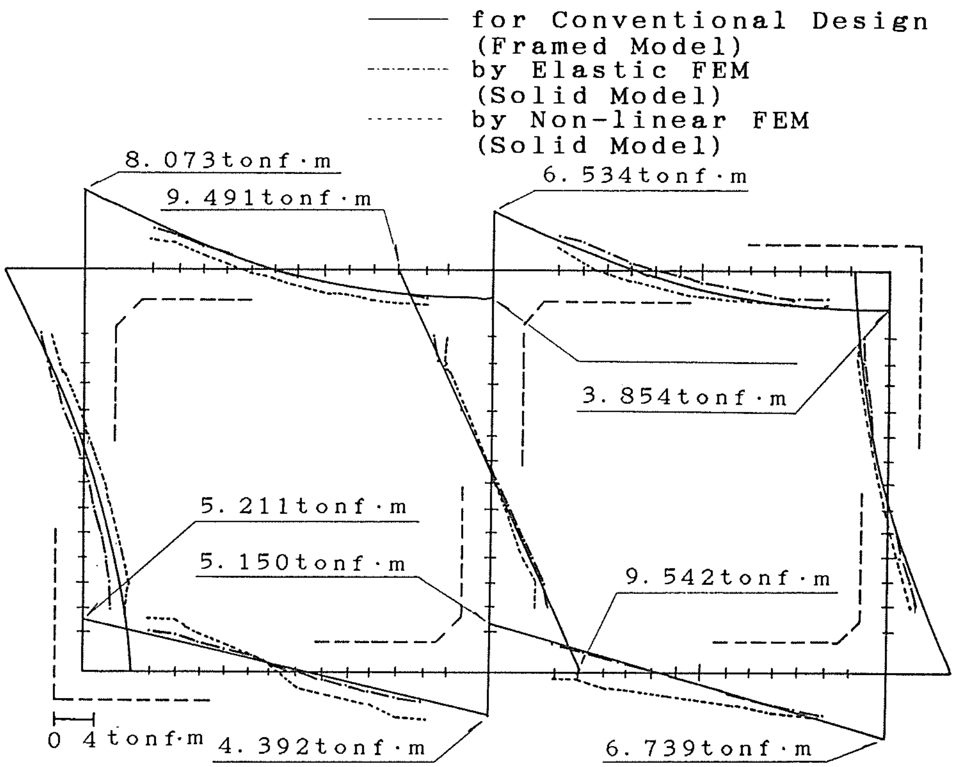

Fig-6 Analytical Results of Bending Moment Distribution