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SEISMIC PERFORMANCE EVALUATION SYSTEM BASED ON THREE-DIMENSIONAL DYNAMIC NONLINEAR FRAME ANALYSIS

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ABSTRACT: *The advance of available computing power at reasonable cost has increased the interest in advanced nonlinear computational methods. At the same time there is an increasing focus on how we wish structures to perform during and after seismic events instead of simply describing the loads to be applied and checking stresses. This has lead to the concepts represented in Performance-Based Seismic Design systems. In this paper we present a Seismic Performance Evaluation System based on the use of a more precise element model that considers material strain-stress nonlinearity. The system fully integrates a 3D dynamic nonlinear analysis and performance checks to meet code requirements. Fiber model beam elements are employed and the strain response is used as a key damage indicator for performance checks.*

KEYWORDS: *3D Dynamic Nonlinear Analysis, Fiber Member Model, Level 2 Earthquakes, Seismic Performance Evaluation Systems, Strain, Damage*

1. INTRODUCTION

Seismic analysis for civil engineering design in high risk seismic zones is moving increasingly towards three-dimensional dynamic nonlinear time history analysis. The Specification For Highway Bridges Part V. Seismic Design (SFHB-V)[1] published in Mar. 2002 by the Japanese Road Association clearly recommends dynamic nonlinear time history analysis methods. However, until now the nonlinear analysis of bridge frames for Level 2 Earthquakes (A strong earthquake level defined by SFHB-V) was limited to the "member nonlinear" models such as force-displacement or moment-curvature methods, e.g. the Bilinear Model or the Takeda Model. These models give at best rough approximations to the stress-strain response for the materials in the element. These models are developed from uniaxial experiments and generally do not consider the interaction of axial force and biaxial moments. They are thus limited to 2D analyses where the axial force remains a constant and the moments are uniaxial. It is desired to have a nonlinear model that not only reflects the material nonlinear characteristics but also gives the simultaneous interactive effects of a multi-directional response.

The fiber element model [2] has recently become popular for simulating the response of an

RC member at the material strain-stress level. It assumes that cross sections remain plane and that the section is divided into a mesh of cells. The cells may vary in size and each cell is assigned a material that defines the hysteretic stress-strain response for that cell. This basic data model permits the balancing of the simultaneous axial forces and biaxial moments with the internal section forces. The internal section forces are calculated as the discrete integrals of stress multiplied by cell area over the section. Member nonlinear models cannot model this interaction as well as a fiber model.

The concept of Performance-Based Seismic Design (PBSD) was also recently introduced to Civil Engineering. For bridge seismic design, the newest SFHB-V gives a detailed description about the permissible damage states for each part of a bridge. Required performances correspond to different levels and types of earthquake. For example, the reusability of a pier is determined by its state classification (Elastic, Low Plastic or High Plastic).

In this paper we present an integrated system that permits fast evaluation of required seismic performance. It combines the results of a 3D dynamic time history material nonlinear analysis with seismic performance definitions and produces a quick to use seismic analysis and design system. A sample model analysis is included.

2. SUMMARY OF SEISMIC PERFORMANCE EVALUATION SYSTEM

The Seismic Performance Evaluation System presented here is composed of a 3D dynamic nonlinear analysis program based on the fiber member model and performance evaluation functions. Their details are shown in the following.

2.1 Three Dimensional Dynamic Nonlinear Method

The employed analysis software [3] was developed by Prof. Goto and Prof. Ijima of The Department of Urban Engineering at Saga University. The original research was directed at geometrical nonlinear analysis of large displacement structures and later the effects of material nonlinearity were also included. The fiber member model was employed as the nonlinear member and its material constitutive law was established with reference to SAKIMOTO[4] and SFHB-V[1]. The comparison of experimental data to calculation results demonstrates good precision for nonlinear structural analysis.

2.2 Seismic Performance Classifications and Their Evaluation

In general, the seismic performance checks are done according to the ultimate state of the member responses. For example, SFHB-V gives the required seismic performances for piers and superstructures as shown in Table 1. However, its checks are done using the curvature or rotation angle from the member nonlinear model analysis. If the material nonlinear hysteretic model is used, the same checks can be done more accurately at the strain-stress level and incorporate the effects of all simultaneous actions.

Strain response of materials can give a more accurate damage evaluation than using the forces acting on sections. Inspection of the cell strain responses quickly reveals the damage patterns in the section. Thus the response of the section can be clearly understood and the design modified to produce the desired behavior.

Table 1. Seismic performance of bridges and ultimate states of members[1)]

	Level 1 Seismic Motions	Level 2 Seismic Motions	
	SP 1*	SP 2	SP 3
Performance Intention	No damage to Bridge Normal Performances	Only Limited Damage occurring, Recovery can be done quickly	No Fatal Damage Occurring to Bridge's Main Function
Piers	Material Response Locating in Elastic Range	Easily Recovered, Limited Damage States	First Entering, Declining State of Horizontal Capacity
Superstructures		Keep in Material Elastic Range or only Secondary Plastic Response occurs	

*SP: Seismic Performance

The system does the safety checks according to the formulae (1)~(3) below. The residual deformation check is omitted to reduce the size of this paper and only the RC pier material damage checks are conducted.

SP 1 : $\varepsilon s \leqq \varepsilon ya$ and $\varepsilon cA' \leqq \varepsilon ca'$ ………… (1)
SP 2 : $(\varepsilon s \geqq \varepsilon y$ and$)$ $\varepsilon cA' \leqq \varepsilon ca2'$ ………… (2)
SP 3 : $(\varepsilon s \geqq \varepsilon y$ and$)$ $\varepsilon cB' \leqq \varepsilon ca2'$ ………… (3)

In which,

εs : Tensile strain of reinforcement
εy : Tensile yielding strain of reinforcement
（＝Compressive strain of reinforcement equivalent to allowable stress strength ）
$\varepsilon cA'$: Compressive strain of concrete at the cover position
$\varepsilon cB'$: Compressive strain of concrete at the most outside reinforcement position
εya : Allowable strain of reinforcement for Level 1 Seismic Motions
$\varepsilon ca'$: Allowable strain of concrete for Level 1 Seismic Motions
（=Compressive strain of concrete equivalent to allowable stress strength ）
$\varepsilon ca2'$: Allowable strain of concrete for Level 2 Seismic Motions
（＝Peak strain at the maximum compressive concrete stress ）

When unconfined concrete exceeds its peak strain, its strength will quickly deteriorate. Hence it is assumed in this paper that SP 2 is required for the cover concrete and SP 3 is required for the core concrete.

The user can input the material strain check values thus permitting specific performance requirements to be defined in a flexible manner.

The system calculates the maxima strain for each material in the section and uses these for the performance checks defined in formulae (1)~(3) above. The check results are shown graphically and the seismic performance of the whole bridge can be easily verified visually.

3. APPLICATION EXAMPLE

3.1 Model

The sample bridge is a 200m structure curved in plan view with 20-30m high piers. Dynamic nonlinear analysis is required for its seismic design under SFHB-V. Figure 1 shows a plan and cross section of the piers and superstructures of the model. Figure 2 shows the model in solid member form. Fiber elements are only used in the expected nonlinear locations in order to form a viable nonlinear mechanism under a large seismic event. Other elements are analyzed as elastic frame members, and these may also be checked to ensure they did not exceed elastic limits, thus verifying the nonlinear mechanism chosen. The bottoms of the footings are fixed and the ends of the superstructure are treated as simply supported. The hysteresis of the core concrete and reinforcement of the pier are displayed in Figure 3. The damage criteria in terms of strain are defined and shown under the stress-strain diagrams by horizontal color bars. An elastic analysis is undertaken first to determine the mode shapes and frequencies for use in the dynamic analysis in conjunction with damping. Figure 4 shows the first 6 modes and frequencies.

3.2 Design Performance and Input Earthquake Records

The target performance is chosen as Seismic Performance 2 under the Level 2 Seismic Motion category. The Hyogo-ken Nanbu Earthquake records with 85% declining are used as the input acceleration (Figure 5(a), (b)) (Last time: 12sec., Interval: 0.02 sec.). The maximum acceleration, 698cm/s^2, occurs at step 208 in the NS component. Acceleration signals are input for the longitudinal bridge axis (EW component) and the transverse direction (NS component) simultaneously. Vertical accelerations may also be input but are excluded here.

3.3 Results and Damage Display

a) Result Display and Damage Check

After dynamic nonlinear analysis, there is a large amount graphical or numerical data available. Figure 6 shows a graph of the deformed shape of the bridge, while Figure 7(a) and (b) give the displacement results of the node at the top of P2, one for the time history response and the other for the displacement in the XZ plane. If needed, a residual displacement check can be done using these results.

Figure 8 presents the damage states and the basic performance check results of this model. Figure 8(a) is a solid display using color to indicate areas of damage and their level. Figure 8(b) represents the same information as 8(a) but shows the fiber sections instead. Figure 8(a) and Figure 8(b) both show the worst damage state that has occurred up to the currently indicated step (208 in the figures). This is referred to as the cumulative damage. The damage states (Heavy, Light or None) are shown by color and these depend on the material type. In addition to the cumulative damage results, the instantaneous damage state at any step is also available. The colorful and intuitive graphical damage results demonstrates the system’s flexibility and usability.

b) Fiber Member Results

The fiber section results permit a more specific investigation into the nonlinear results including the location of damage in the section and the time step. The materials stress-strain history can also be animated graphically. Figure 9 summarizes the results of a fiber section. The fiber element has stresses and strains calculated at two internal points. Figure 9(a) is a display of the cell damage incorporating the section location in the member. Figure 9(b) is a plane display of the same information for one calculation point. In Figure 9(b), the user may also select a cell and display that cell's hysteresis path. This is shown in Figure 9(c) for reinforcement and Figure 9(d) for concrete. The system can also give all basic force-displacement results as per other member nonlinear analysis tools. Figure 9(e) is the M-phi results of the section plane around the y_p axis and Figure 9(f) is the M-phi results of the section plane around the z_p axis.

4. CONCLUSIONS

We draw the following conclusions about the above Seismic Performance Evaluation System:

- The proposed system is flexible and easy to use to evaluate with confidence the overall performance of a structure subject to a seismic event of any size. Individual member checks are also conducted. The results give a clear indication of the response of the structure and the graphics are plentiful and intuitive.
- The fiber member model accounts for the interaction of axial and flexural actions in the member more accurately than member nonlinear methods. The fiber method is used to develop and verify other simpler member nonlinear methods. With the advance of computing power, it can now be directly used in the main analysis and avoid the shortcomings of member nonlinear models.
- The nonlinear analysis method based on the Fiber Member Model not only gives the material level response of structures for use in seismic performance evaluations, but also presents the force-displacement responses like moment-curvature.

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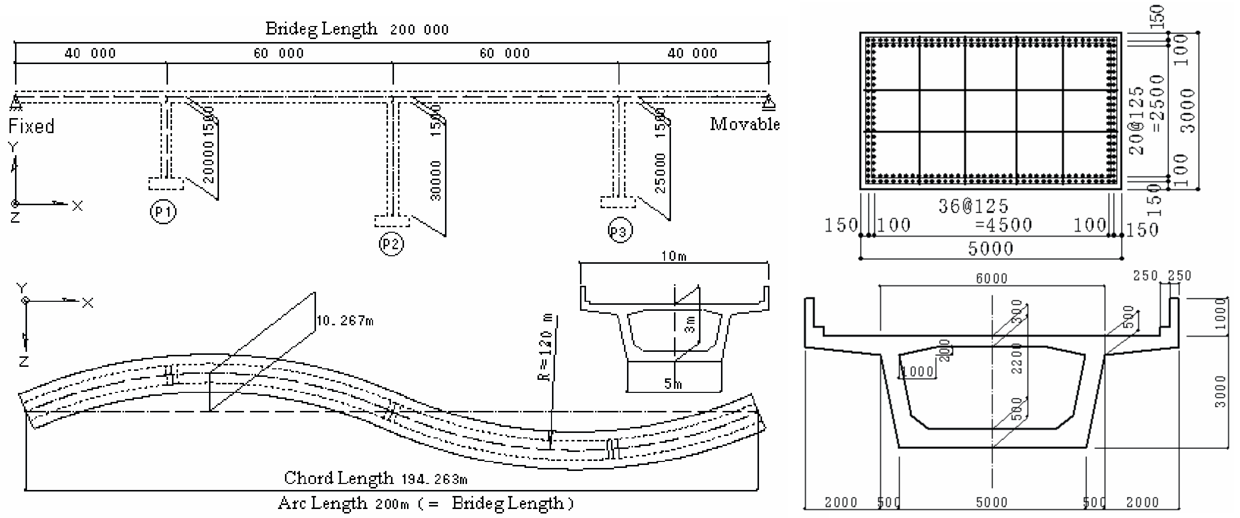


(a) Plane maps

(b) Cross sections of piers and superstructures

Figure 1. A 4 span rigid frame curve bridge

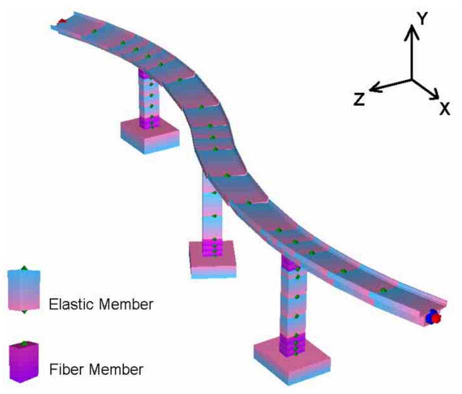
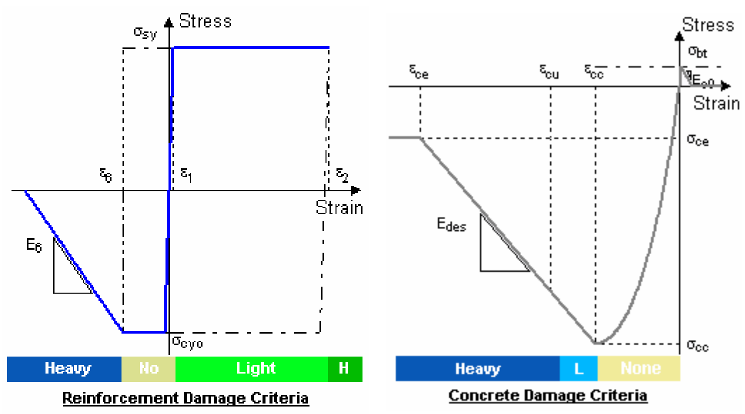


Figure 2. Analysis model



(a) Reinforcement of piers

(b) Core concrete of piers

Figure 3. Material hysteresis and damage definition

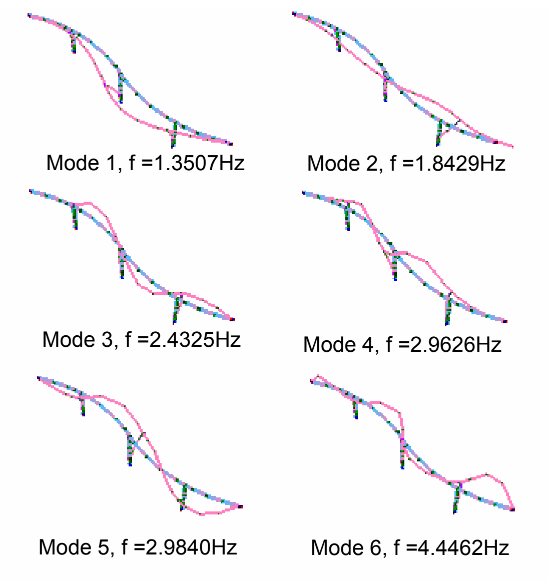
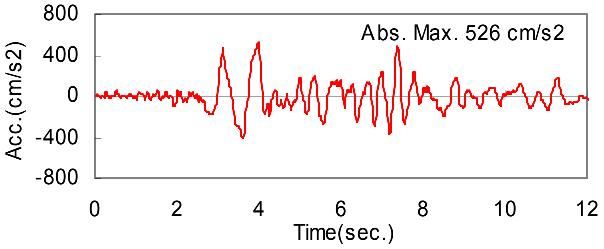
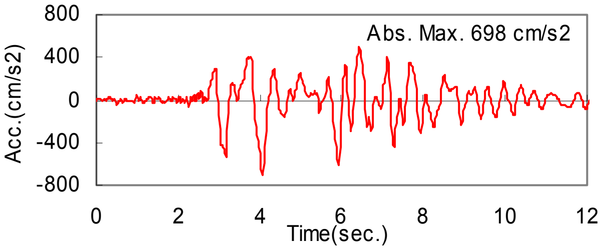


Figure 4. Natural vibration modes



(a) EW Component



(b) SN Component

Figure 5. Input earthquake accelerations

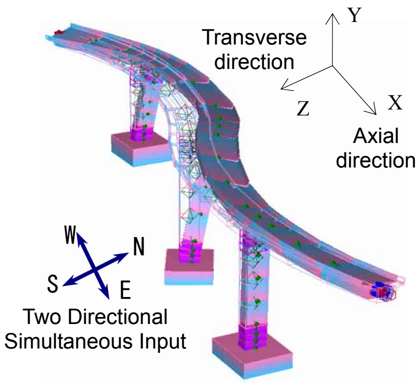
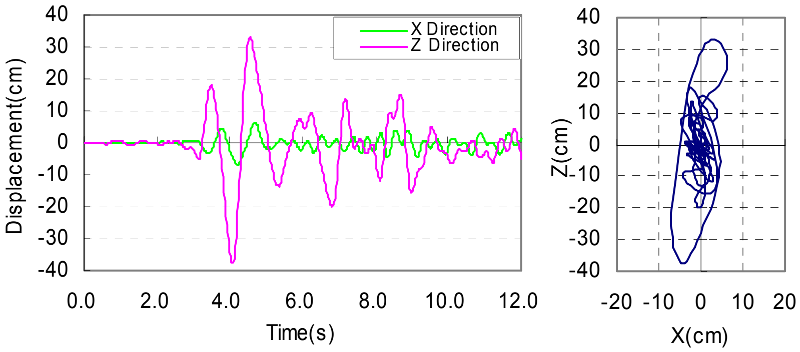


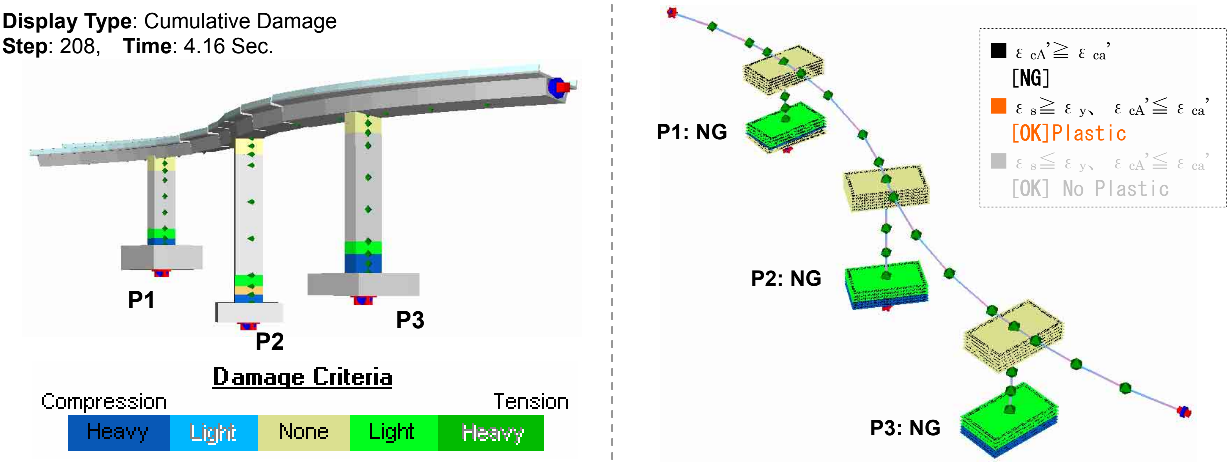
Figure 6. Deformed graph of model



(a) Displacement time history

(b) Trace in X-Z plane

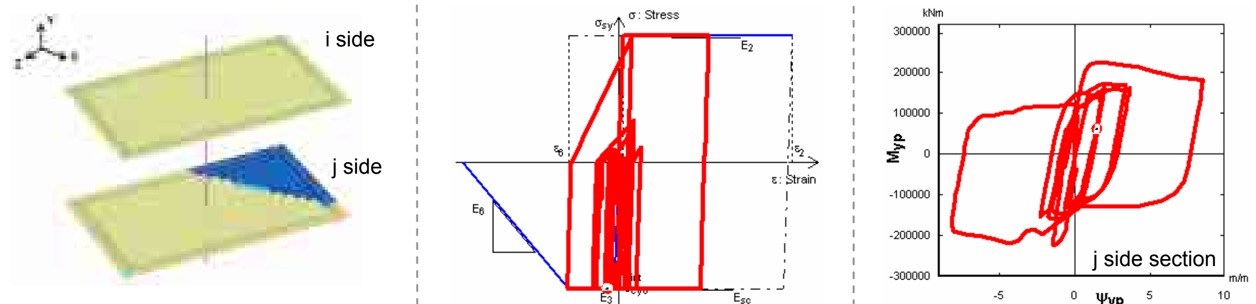
Figure 7. Node displacement response P2 top



(a) Damage member display in solid

(b) Damage member display in fiber sections

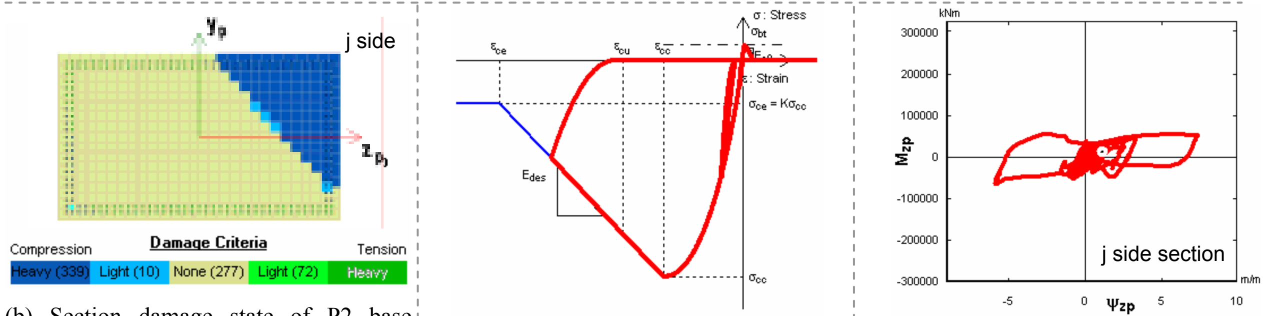
Figure 8. Damage results by two directional earthquake input



(a) Section damage state of P2 base section at step 208 (in 3D)

(c) Stress-strain history of steel cell

(e) M-φ response of section around y_p



(b) Section damage state of P2 base section at step 208 (in plane)

(d) Stress-strain history of concrete cell

(f) M-φ response of section around z_p

Figure 9. Fiber member results