Seismic Damage Evaluation of a Conventional Highway Bridge

Evaluation des dégâts causés par un séisme
sur un pont-route conventionnel

Seismische Zerstörungsuntersuchung
einer konventionellen Autobahnbrücke

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SUMMARY

As part of a larger project to improve seismic retrofit guidelines for conventional highway bridge structures in the United States, a two-dimensional finite element analysis has been performed for a typical structure configuration. Damage of a vulnerable single-column central pier is modelled using a nonlinear-dynamic hysteretic program, and damage indices are computed based on both hysteretic energy and maximum softening concepts. The interaction of the flexible column with the stiff deck is examined for measured and simulated seismic events.

RÉSUMÉ

Élément d'un large projet visant à améliorer les directives pour les modifications des structures des ponts-routes aux États-Unis, un calcul par différences finies bidimensionnel a été exécuté pour un cas de structure typique. Les dégâts provoqués à un pilier central composé d'une colonne est modélisé en utilisants à la fois des concepts d'énergie hystérétique et de tolérance maximum. L'interaction de la colonne flexible avec le tablier rigide est examinée pour des cas de charges sismiques réels et simulés.

ZUSAMMENFASSUNG

Als Teil eines grösseren Projekts, die seismische Richtlinie für konventionelle Autobahnbrücken in den Vereinigten Staaten zu verbessern, wurde eine zweidimensionale Finite-Element-Analyse für einen typischen Bauwerk ausgeführt. Schäden für eine verwundbare Einzelsäule, bestehend aus einem zentralen Pfeiler, ist ausgeführt mit einem "non-dynamic hysteric" Programm, und Schadenindexe sind basierend auf "hysteric" Energie berechnet und auch auf maximale Erweichungskonzepte. Die Wechselwirkung von der biegsamen Säule mit dem steifen Deck ist für gemessene und simulierte seismische Vorfälle untersucht worden.

1. INTRODUCTION

Important modes of seismic damage have been identified from a literature review [1] of observed damage to existing structures in the United States constructed in the last 25 years. With a few exceptions, all major damage occurred in continuous-span, prestressed or reinforced concrete, multi-cell box-girder bridges having one of three basic configurations:

1. Curved multi-span, single-column bent
2. Skewed short-span, multi-column bents
3. Straight double-deck viaducts with hinged columns

In this study, we focus on the simplest form of the first type, a straight two-span overpass with a single-column bent. In particular, we select the Meloland Road Overpass (MRO), located in El Centro, California, which has been instrumented extensively enough to monitor the primary global vibration modes. A schematic view of the MRO configuration and the instrumentation layout is shown in Figure 1. The MRO has been reported by Werner *et al* [2] for a nondamaging event of M_L= 6.4. Our goal is to calibrate a dynamic model to the undamaged state and then to simulate damaged states using hysteretic models. Unfortunately, calibration against observed damaged states is not yet possible for lack of data.

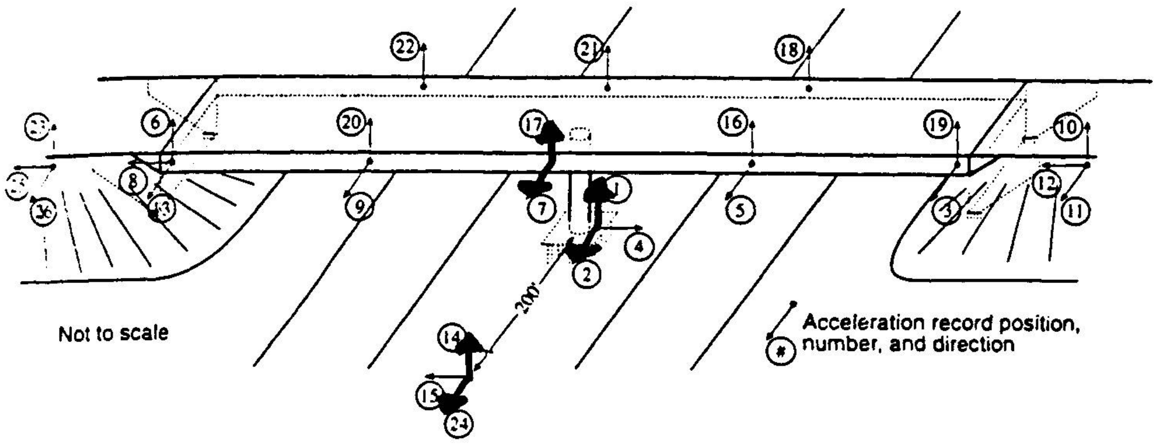

Fig. 1 Schematic MRO configuration and instrument array

The literature review has indicated that, broadly speaking, damage is concentrated in columns and their connections to footings and cap-beams. Failure occurs at varying heights of the column and in both short and tall columns. Combinations of bending and shear failure modes are apparent in many cases with axial force contributing to the damage mode. Shear failures at midheight also occurs at the end of flared sections of short columns.

The factors affecting damage modes have both demand and capacity aspects. Key capacity factors have been identified by Chai *et al* [3] who point to the role of reinforcement detailing and the effects of confinement on strength of concrete. Simplified procedures for capacity and demand analysis have been proposed based on static or pseudo-static procedures. These procedures have recently been applied in failure assessments of bridges damaged during the Northridge earthquake. Priestley *et al* [4] provide moment-curvature and shear-strength analyses and Buckle *et al* [5] compute vulnerability ratings to show the expected distribution of damage in different locations of a structure.

The pseudo-static procedures, however, do not adequately account for the complex influence on column demands that occur dynamically in the presence of damage. The relationship between relative stiffness, mass distribution, restraints at supports and connections, and global modes of vibration, need closer examination so that the key parameters may then be identified. Calvi *et al* [6] attempt this but with design in mind rather than retrofit. Our study attempts to examine the above relationships for the MRO column/deck system using a simplified representation of the structural dynamic system.

2. IDARC MODEL OF THE MRO

The most vulnerable region of the MRO is the central pier which consists of a single pile-supported column and a solid beam made integral with the box-girder deck. Figure 2 shows the basic configuration taken from as-built drawings and the IDARC beam-column element idealization. Also shown are the locations where the participating deck mass has been lumped and where accelerations have been monitored. For simplicity, the base is fixed and the measured horizontal and vertical accelerations are applied as inputs. The rotational flexibility of the footing and piles is therefore neglected. This flexibility will tend to lower the natural frequency

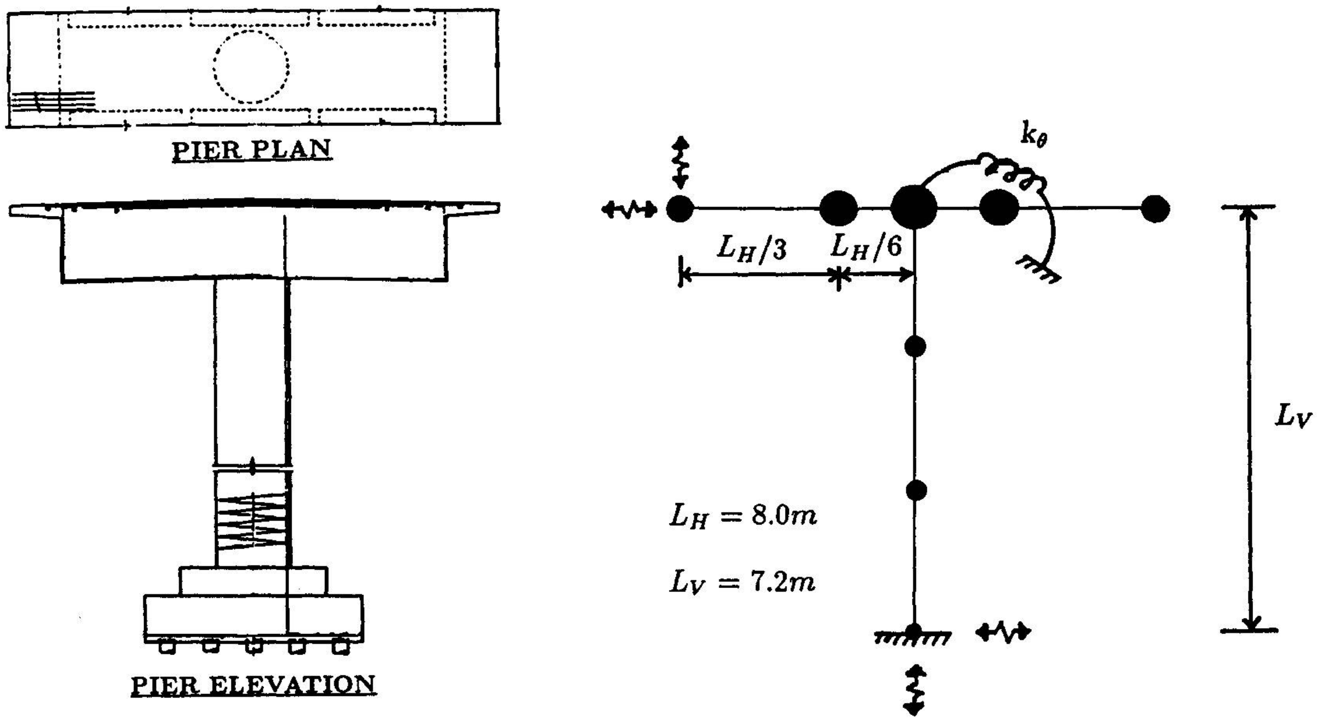

Fig. 2 IDARC model of single-column central pier

of the system relative to that of the fixed base model. All geometric data were selected to be consistent with the as-built drawings and the in-situ values of material properties reported in Werner *et al* [7]. Key data for the model are summarized in Table 1 which indicate relative values of stiffness, rigidity, and bending moments.

Description	Quantity	Value or Ratio
deck rot. stiffness	$k_\theta = (GJ/L)_{eq}$	11.29 GN-m/rad
deck lat. stiffness	$k_{HD} = c_H\ (EI/L^3)$	175.1 MN/m
col. flex. rigidity	EI_C	7.314 MN-m^2
col. yield. moment	M_{YC}	+/−121.2 kN-m
dead load reaction	$P_0 = c_V\ (wL)$	5.338 MN
meas. freq.-1st transv.	f_{1T}	2.47 Hz
col. lat. stiffness	k_{HC}/k_{HD}	0.5
col. crack. moment	M_{CC}/M_{YC}	+0.26 /−0.26
capbm. flex. rigidity	EI_B/EI_C	1.05
capbm. yield. moment	M_{YB}/M_{YC}	+0.18 /−1.27
capbm. crack. moment	M_{CB}/M_{YC}	+0.18 /−0.25
dead load reaction	$P_0/A_g f'_c$	.0828
model freq. @ W/P$_0$= 0.25	f_1/f_{1T}	1.03
model freq. @ W/P$_0$= 0.50	f_1/f_{1T}	0.85

Table 1 Key data for MRO IDARC model

Interaction of the pier with the deck cannot be neglected. If no interaction is present, the solid capbeam collapses because of the heavy deck masses that are cantilevered. Concentrating all the mass at the beam-column connection is not realistic either. A compromise has been made here by distributing a fraction of the estimated dead load reaction, W/P_0, along the beam with highest concentration at the connection. The distribution and dead load fraction have been selected by tuning the fundamental period to that identified by measurements of Werner *et al* [2].

The torsional and lateral stiffness of the deck have been modeled through a linear rotational spring applied at the beam-column connection. A separate lateral spring is not necessary, because the resisting moment developed by the rotational spring produces shears which resist lateral displacement at the deck level. Beam theory has been used to estimate the spring constant with rigidities corresponding to the ATC-32 recommendations described in Werner *et al.* [7].

3. MODEL CALIBRATED RESPONSE

Time histories were computed of response at the deck level to inputs at the footing level using acceleration records measured [2] at the site during the M_L= 6.4 Imperial Valley earthquake of 1979. Figure 3 shows a comparison of the measured and computed responses. No damage is predicted nor was observed in this case having a horizontal peak ground acceleration, PGAH= 0.32 g. The horizontal response is predominantly in the first two modes shown in Figure 4. The vertical response is predominantly in the third and fourth modes. The 2-D model is capable of representing the horizontal response adequately, but cannot capture the vertical response which is response which is associated with deck behavior that is 3-D in nature.

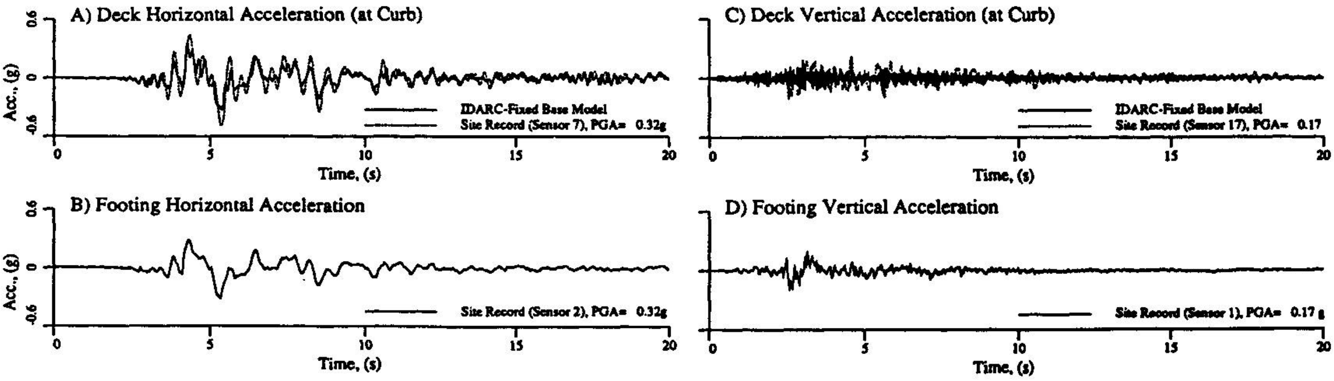

Fig. 3 Response time histories for 1979 Imperial Valley earthquake

4. MODEL DAMAGED RESPONSE

Damage was studied by scaling the records uniformly to higher PGAH. The results were highly sensitive to the assumed participation level of the dead load fraction, its distribution, and the spring constant level. Leaving the spring constant at the value estimated by beam theory and using default values of tri-linear hysteresis parameters set by the program, the remaining parameters with the most uncertainty were the mass level and distribution. A dead load fraction of 1/3 was required to tune the fundamental frequency to the measured value of 2.47 Hz. This fraction is too low to introduce significant damage even at extremely high accelerations. A fraction of 2/3 and higher introduced unrealistically high response accelerations and damage states at moderate input accelerations. A fraction of 1/2 was therefore used in the damage analysis.

Figures 4 and 5 shows two manifestations of the damage from an input event scaled to PGAH= 1g. Figure 4 shows the time-varying softening (increase in) modal periods during the event and Figure 5 the moment-rotation hysteresis. The softening was used to compute a Maximum-Softening (M-S) damage index, $\delta_M = 0.30$. The hysteresis was used to compute a Park-Ang (P-A) damage index, $\delta_{P-A} = 0.40$. Figure 4 also shows the computed variation of the respective indices with PGAH. It should be noted that the upper bound for the M-S index is 1.0.

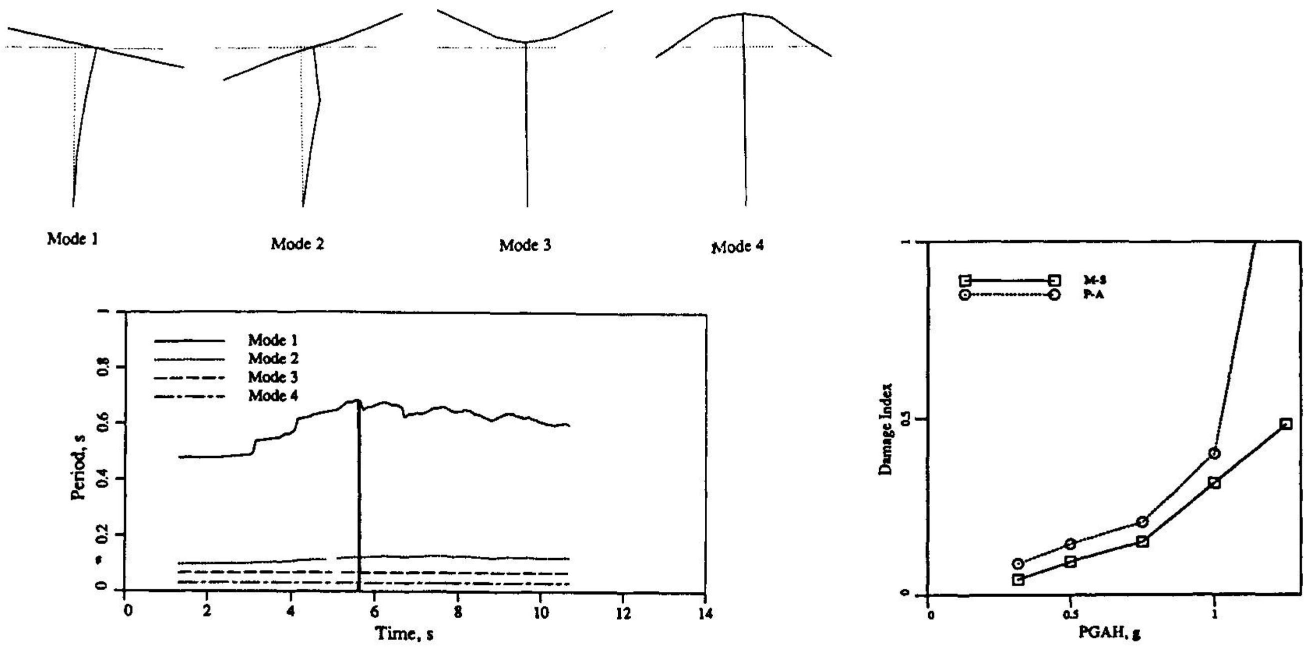

Fig. 4 Modal softening damage for scaled PGAH= 1g event

5. CONCLUSIONS

A simple 2-D analysis of a well-instrumented bridge of simple configuration reveals the complexities of response features governing damage in a nonlinear-dynamic setting. It is clear that 2-D models are not capable of representing damage behavior reliably because of 3-D interaction and global modes of vibration. They may be useful for linear analysis and design based on such analysis, but may be unconservative if used to predict damage without applying considerable judgement to the results. Research is needed to develop efficient and reliable analysis tools and simplified relationships to support damage assessment and retrofit design of existing conventional highway bridge structures. Of particular importance is the modeling of flexure and flexure/shear failure in the presence of realistic representations of axial and biaxial bending forces.

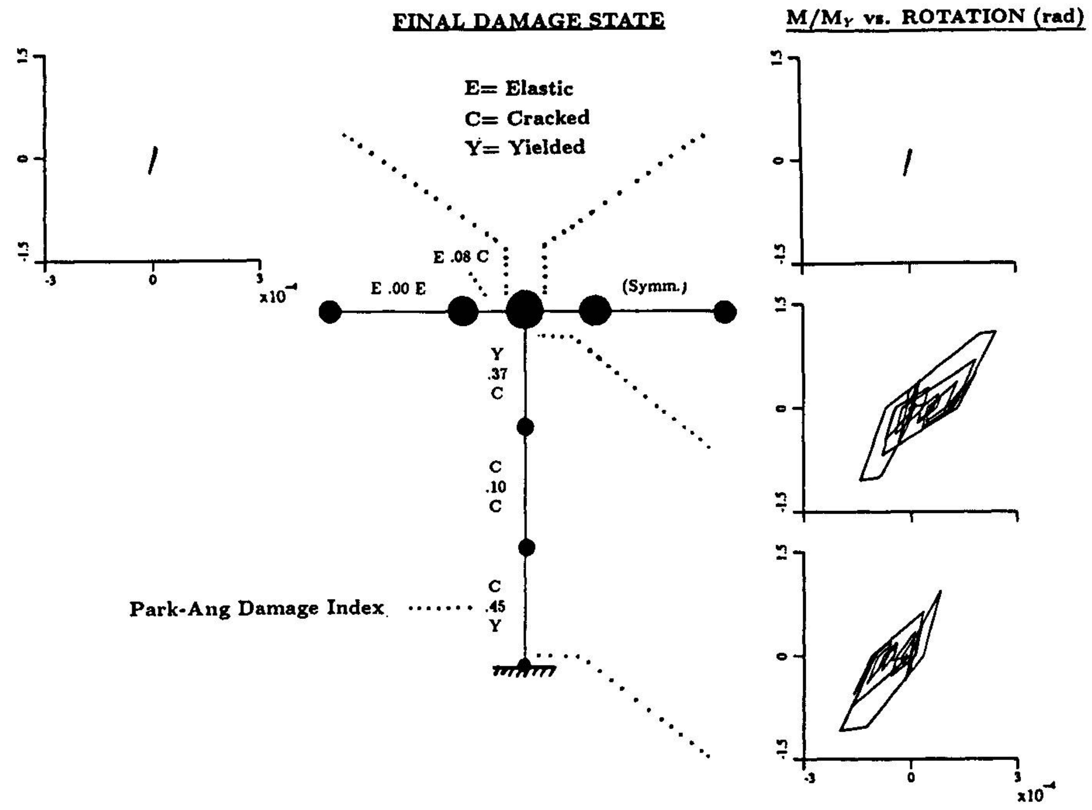

Fig. 5 Hysteresis damage for scaled PGAH= 1g event

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