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MODELING FOR NONLINEAR RESPONSE PREDICTION OF PILE-SUPPORTED STRUCTURES

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SUMMARY

A methodology for modeling of pile-supported structures for earthquake response prediction in the nonlinear regime is presented. Details for modeling for the initial elastic response stage, the concrete cracked stage and the structure yielding stage are treated separately. The modeling details evolved through a detailed evaluation of the response of the Imperial County Services Building during the Imperial Valley Earthquake of October 15, 1979.

INTRODUCTION

A methodology for the nonlinear response prediction of pile-supported structures based on a literature survey, judgment and experience was formulated (Ref. 1). The response data of the Imperial County Services Building obtained during the Imperial Valley Earthquake of October 15, 1979 was extensively used to check all modeling conclusions (the details of this process will be described elsewhere). The methodology was then applied, without any further adjustments, to four pile-supported structures that had undergone a variety of damage states during the September 19, 1985 Mexico Earthquake with very satisfactory and encouraging results (Ref. 2). This paper describes the details of the modeling of both structures and pile foundations for earthquake response prediction in the nonlinear regime. Considering the fact that the ultimate objective of this development was to apply the methodology to Mexico City structures it was imperative that the structure-pile system eigenparameters be obtained in a much more accurate fashion since Mexico City earthquakes are characterized by a relatively narrow frequency band.

MODELING OF STRUCTURES

Two types of models are developed: equivalent elastic models for linear analysis and inelastic models for nonlinear analysis based on four distinct stages of ground motion excitation/response:

1. Initial Response Stage, defined as the initial low seismic excitation.
2. Concrete Cracked Stage, defined as the time period between the end of the Initial Response Stage and the initiation of global yielding.
3. Structure Yielding Stage, defined as the time period between the end of Concrete Cracked Stage and the point at which ground motion begins to subside.

4. Post-Yielding Stage, defined as the excitation after the Structure Yielding Stage. This stage response is not of importance for modeling.

Interest in models for all of these response stages stems from the fact that for critical facilities only minor or no inelastic response is usually permitted.

Initial Response Stage The assumption of rigid floor slabs does not provide acceptable results even for this low excitation level. Out-of-plane flexibility must be included even if slabs can be assumed to be rigid in their own plane. For this Stage concrete does not crack and therefore gross area of the members is used in computing the axial and shear areas and the torsional moments of inertia. The bending moment of inertia for walls, beams and columns is based on transformed uncracked sections which include the reinforcement steel. The section properties for beams includes centerline-to-centerline slabs and floor joists that form T-sections. The calculated shear areas and moments of inertia of beams and columns are then modified to account for the differences between the actual clear lengths of members as the computer models are based on center-to-center dimensions. These modifications are based on

$$A_i = A_c \frac{l_i}{l_c} \;, \quad T_i = T_c \frac{l_i}{l_c} \quad \text{and} \quad I_i = I_c \left(\frac{l_i}{l_c}\right)^3 \quad \text{where}$$

A, T and I refer respectively to shear area, torsional moment of inertia and bending moment of inertia. The subscripts i and c refer to center-to-center and clear spans respectively.

Concrete strengths are based on the best estimate values at the time of construction modified for aging. Modulus of elasticity is calculated according to ACI 318-83 (Ref. 3).

Concrete Cracked Stage Two modifications are introduced to the uncracked section results for beams described above: the effective flange width of T-sections is reduced according to ACI 318-83, Section 8.10 and the effective moment of inertia for beams is calculated according to ACI 318-83, Eq. 9-7. Additionally, the mid-span and end-span moments of inertia are averaged.

In computing the moment of inertia of columns and shear walls, the neutral axis of a transformed section is assumed to be at the centerline of the cross section, as suggested in Ref. 4, to account for the effects of axial load. Other section properties are calculated the same way as for the Initial Response Stage including the modifications to account for clear span lengths.

Structure Yielding Stage (Inelastic Model) The use of a stick lumped mass model for the inelastic analysis of multi-story structures is not a simple problem. As a minimum a two-dimensional (2-D) model is necessary. Obviously a three-dimensional (3-D) model is prohibitive for large problems. The section properties and moment capacities of each individual member in the 2-D model are the sum of the respective properties of all members from the 3-D Concrete Cracked Stage model. In order to account for the dead load on the structure, fixed end forces and moments are applied on the horizontal beam elements. In computing the bending moment capacities of columns, the axial loads on the columns are obtained from the 3-D Concrete Cracked Stage model for dead load and estimated seismic loads (using a simplified response spectrum analysis) in two orthogonal directions (x + 0.4y). To account for the two-directional seismic input four interactive diagrams at 30°each is required (see Fig. 1). Best estimate parameter values are used.

All structural elements are represented by degrading stiffness beams (extended Takeda model). The parameters of this model are shown in Fig. 2. The extended Takeda model includes: (1) a reduction of the unloading stiffness by an amount which depends on the largest previous hinge rotation, and (2) a variable reloading stiffness, which is larger than that of the Takeda model and also depends on the past rotation history. These modifications of stiffness are controlled by the stiffness degrading parameters, α and β (see Fig. 2 for the definition of α and β). For Mexican practice the stiffness degrading parameters, α and β, for all horizontal beams were selected as 0.4 and 0.25, respectively; for all column members α and β were selected as 0.5 and 0.2, respectively. The strain hardening ratio for inelastic beam-end-moment rotation relationship for all structural members was chosen as 0.01.

MODELING OF FOUNDATIONS

For response calculations pile foundations are characterized by their impedances. The impedances are calculated by the computer program DYNA (Ref. 5). Table 1 summarized the foundation impedances output from DYNA for all degrees-of-freedom including coupling terms. Impedances are frequency-dependent. However, considering the fact that nonlinear structural analysis codes at present cannot deal with frequency-dependent impedances, a frequency-independent set of impedance coefficients are selected by an iterative process. Two bilinear truss elements are used to represent the nonlinear impedance functions.

The damping results obtained from DYNA are increased three times; however, the group efficiencies after this modification would not exceed 50 percent. The justification for these modifications are a series of studies, review of available data and judgment (Ref. 6).

<u>Initial Response Stage</u> The foundation impedances for this stage of response are based on low-strain soil properties obtained from in-situ soil shear wave velocity measurements.

<u>Strong Shaking Stage</u> Structural response beyond the Initial Response Stage may result in: a) moderate to large levels of strain in the soil surrounding the piles, in which case the effects of nonlinear soil behavior on the stiffness and energy dissipation characteristics of the pile-soil system is considered, b) cracking of pile concrete section to a certain depth into the soil. These effects are treated approximately and Fig. 3 shows the steps necessary to arrive at the pile group impedances. Two codes are used in the calculations, COM624 (Ref. 7) and DYNA (Ref. 5).

Starting with the pile data, soil profile and estimated horizontal and axial loads, COM624 determines the static stiffness and bending moment as a function of depth. The behavior of the soil surrounding the laterally loaded pile is described in terms of p-y curves which relate soil resistance to pile deflection at various depths below the surface. In general, p-y curves are nonlinear and depend on several parameters, including soil pressure, soil shear strength and number of loading cycles. The weak zone capability of DYNA code (Ref. 8) is then used to calculate the nonlinear (dynamic) impedances of either a single pile or a group of piles.

As summarized in Fig. 3, there are five main steps to the procedure:

<u>Step 1</u>. Starting with the uncracked pile, calculate single pile horizontal stiffness using COM624. Soil nonlinearity is automatically considered. If the induced moments along the pile are less than the associated cracking moments, M_{cr}, proceed to Step 2. Otherwise calculate the extent of cracking along the pile, adopt effective moments of inertia for these segments and calculate the cracked single pile horizontal stiffness. A sample result of these calculations

is shown in Fig. 4. It is seen that nonlinear soil effects (p-y curve) begin to affect pile lateral stiffness for lateral loading of the pile head in excess of about 28 KN. At about 46 KN cracking in the pile begins which further reduces the lateral stiffness. For an estimated equivalent static load of 133 KN the lateral stiffnesses of a single pile are approximately one-third and one-fifth of the value calculated by the low-strain DYNA approach, for uncracked and cracked pile, respectively.

Step 2 Single pile low soil strain DYNA analysis using either the uncracked or cracked pile properties from Step 1.

Step 3 For either the uncracked or cracked models, compare the results from Steps 1 and 2 to determine if significant nonlinear soil response is expected. If no significant nonlinear response is predicted, proceed to calculate pile group effects using DYNA. Otherwise proceed to Step 4.

Step 4 Prepare DYNA weak zone model. Soil nonlinearity can be considered in DYNA by means of a weakened cylindrical zone around the pile (Ref. 8). Soil nonlinearity is accounted for by a reduced shear modulus and increased material damping of the inner region.

CONCLUSIONS

Based on the recorded response of the Imperial County Services Building obtained during the Imperial Valley Earthquake of October 15, 1979, a set of modeling rules for structures and piles is developed for each of three distinct response stages during strong ground motion excitations. The rules were applied to four pile-supported structures that suffered quantifiable damage during the Sept. 19, 1985 Mexico Earthquake with very satisfactory and encouraging results.

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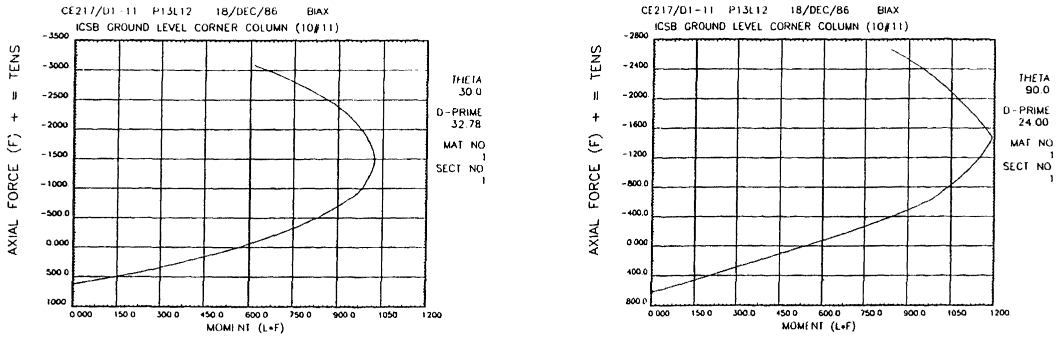


Figure 1 SAMPLE COLUMN INTERACTION DIAGRAMS

Frequency (Hz)	0.5	1.0	1.5	2.0	3.0	4.0	6.0	8.0
KXX(KN/m x 10^6)	3.85	3.85	3.85	3.85	3.85	3.85	3.85	3.85
KYY(KN/m x 10^6)	4.12	4.12	4.12	4.12	4.12	4.13	4.13	4.13
KWW(KN/m x 10^6)	5.40	5.40	5.40	5.40	5.40	5.40	5.40	5.41
KPPX(m-KN/rad x 10^9)	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
KPPY(m-KN/rad x 10^9)	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74
KZT(m-KN/rad x 10^9)	1.84	1.84	1.84	1.84	1.84	1.84	1.84	1.84
KXP(KN/rad x 10^6)	-7.41	-7.41	-7.41	-7.41	-7.41	-7.41	-7.41	-7.42
KYP(KN/rad x 10^6)	-6.91	-6.91	-6.91	-6.91	-6.91	-6.91	-6.92	-6.92
CXX(KN-s/m x 10^4)	1.51	1.19	1.08	1.03	0.975	0.949	0.922	0.908
CYY(KN-s/m x 10^4)	1.62	1.27	1.16	1.10	1.05	1.02	0.988	0.973
CWW(KN-s/m x 10^4)	2.65	1.72	1.41	1.26	1.11	1.03	0.951	0.912
CPPX(m-KN-s/rad x 10^6)	7.17	4.66	3.83	3.41	2.99	2.78	2.57	2.47
CPPY(m-KN-s/rad x 10^6)	13.44	8.74	7.17	6.39	5.61	5.21	4.82	4.62
CZT(m-KN-s/rad x 10^6)	7.24	5.69	5.17	4.91	4.65	4.52	4.39	4.32
CXP(KN-s/rad x 10^4)	-2.95	-2.25	-2.02	-1.91	-1.79	-1.73	-1.68	-1.65
CYP(KN-s/rad x 10^4)	-2.75	-2.10	-1.89	-1.78	-1.67	-1.62	-1.56	-1.54

KXX - Horizontal Translational Stiffness (x-direction)
KYY - Horizontal Translational Stiffness (y-direction)
KWW - Vertical Translational Stiffness
KPPX - Rotational Stiffness (rotation about x-axis)
KPPY - Rotational Stiffness (rotation about y-axis)
KZT - Torsional Stiffness
KXP - Cross Stiffness (x-direction)
KYP - Cross Stiffness (y-direction)

CXX - Horizontal Translational Damping (x-direction)
CYY - Horizontal Translational Damping (y-direction)
CWW - Vertical Translational Damping
CPPX - Rotational Damping (rotation about x-axis)
CPPY - Rotational Damping (rotation about x-axis)
CZT - Torsion Damping
CXP - Cross Damping (x-direction)
CYP - Cross Damping (y-direction)

Table 1 SAMPLE FOUNDATION IMPEDANCES FROM DYNA OUTPUT

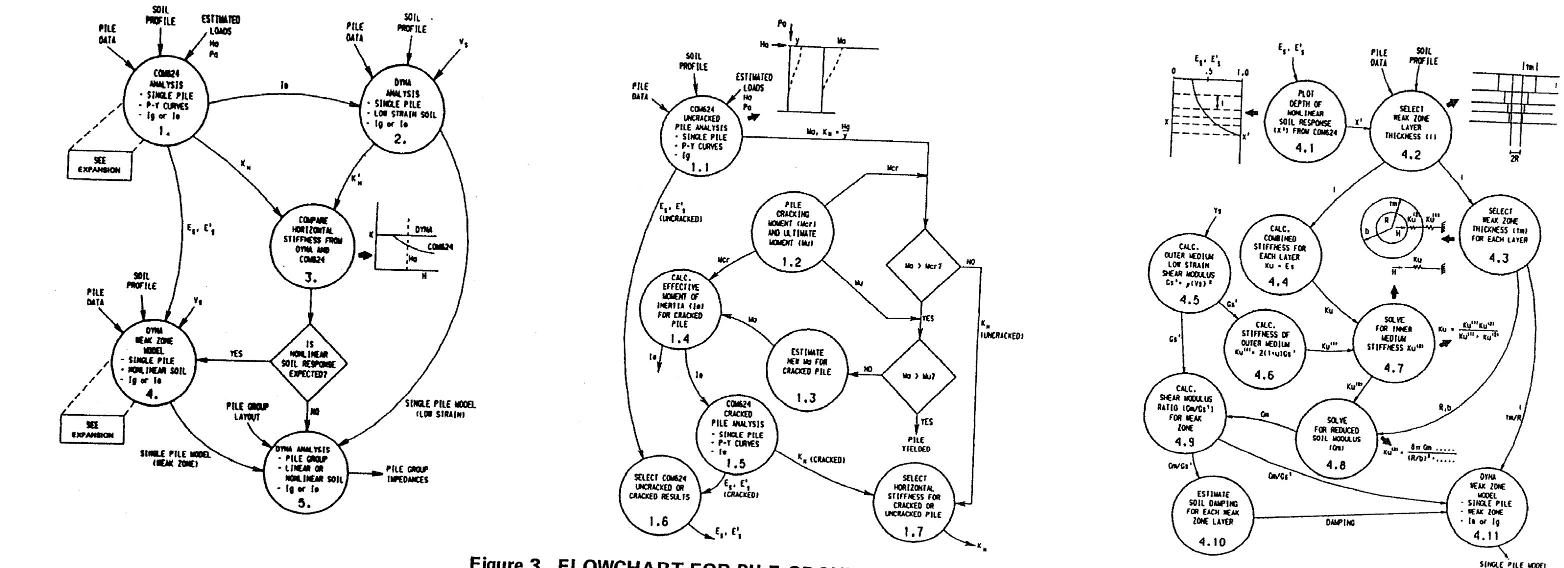
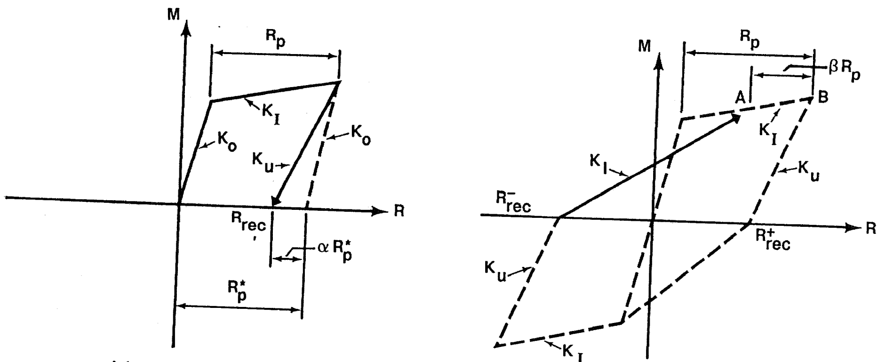


Figure 3 FLOWCHART FOR PILE GROUP IMPEDANCE CALCULATION



(a) Unloading Stiffness

(b) Reloading Stiffness

Figure 2 HINGE MOMENT–ROTATION RELATIONSHIP FOR EXTENDED AKEDA MODEL

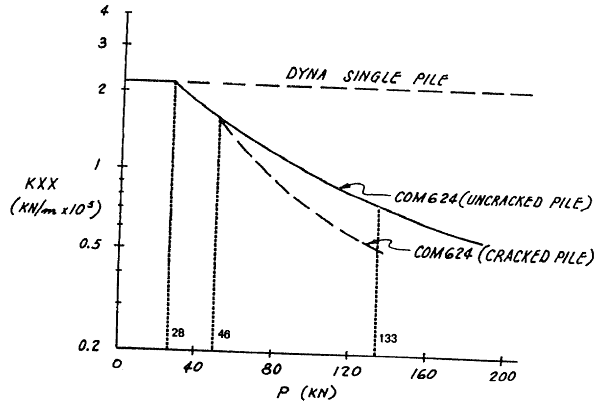


Figure 4 PILE HEAD STIFFNESS (KXX vs. LATERAL LOAD (P))