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ABSTRACT

This paper discusses seismic soil-structure interaction (SSI) analyses for a 10-story office building with three levels of basement (10 meters) located in downtown Oakland, California. The objectives of these analyses were to assess the effects of SSI on the response of the building and to develop ground-level input earthquake motions at the base of the building for use by the project structural engineer. The SSI analyses were conducted using the two-dimensional finite element program FLUSH. The results of these analyses indicate that SSI has a negligible effect on horizontal ground motions at and near the building's predominant period (T ~ 1.8 seconds). Minor but unfavorable SSI effects were found at higher frequencies. The effects of SSI on the vertical motions in the building were for practical purposes, negligible. Parametric studies indicate that more favorable SSI effects may have been realized if the building was surrounded by softer soils.

INTRODUCTION

Research on the effects of seismic soil-structure interaction (SSI) has led to important advances in the state of engineering practice (e.g. SSI provisions of the Applied Technology Council [1978] and NEHRP [1997] codes). Despite these advances, SSI effects continue to be ignored in most dynamic analyses of structures. In some cases neglecting SSI is warranted, as its effects on some buildings are for practical purposes, negligible. In many other situations however, SSI can significantly alter a structure's response to ground motion. When SSI affects a structure, it is often beneficial, with ground-level motions in the structure less intense than those in the adjacent free field. The opposite effect may occur in other situations, with stronger ground motions developing in the structure then in the adjacent free field. Whether SSI effects are beneficial or disadvantageous primarily depend on a building's structural properties and surrounding geologic media. A project-specific evaluation must be performed to determine if SSI effects are beneficial or detrimental to the seismic performance of a structure.

This paper discusses SSI analyses for a 10-story office building with three levels of basement (10 meters) located in downtown Oakland, California. The objectives of these analyses were to assess the effects of SSI on the office building and develop ground-level input earthquake motions in the building for use by the project structural engineer. This paper is not intended to provide an overview of SSI analyses, but rather to document a project-specific SSI analysis, provide insight into the relative importance of these effects, and offer some guidance to practitioners and researchers in conducting other project-specific SSI analyses. Concise overviews of SSI may be found in Kramer (1996), ERPI (1991), and Stewart (1999). A comprehensive treatment of this phenomenon is presented by Wolf (1985).

PROJECT BACKGROUND

The East Bay Municipal Utilities District (EBMUD) Administration headquarters building was designed in 1986 in accordance with the City of Oakland-adopted 1982 Uniform Building Code (UBC) [ICBO 1992]. The building, shown in Figure 1, is rectangular in plan (90 m by 33 m), and contains 23,000 m^2 of aboveground office space, and 14,000 m^2 of belowground parking. The aboveground portion of the building structure consists of a 10-story welded steel moment frame. The building contains three levels of belowground parking in a 10 m deep basement, which consists of concrete columns, shear walls and floors. The structure is supported by a 1 m thick mat foundation. A seismic safety assessment conducted in 1998 concluded that the building was in compliance with the building code for which it was designed, but that observations from the 1994 Northridge Earthquake suggested that the building could sustain significant damage during a major earthquake on the nearby Hayward Fault.

Figure 1. The East Bay Municipal Utilities District (EBMUD) Administration headquarters building.

The seismic safety assessment report recommended that the building be upgraded to meet life safety performance levels consistent with the goals of EBMUD's system-wide Seismic Improvement Program. The report presented two alternative seismic upgrade schemes, and suggested that a non-linear structural dynamics analysis be conducted to further evaluate these upgrade schemes.

A second phase of the study began in 1999, and included detailed structural analyses of the seismic upgrade alternatives developed during the initial phase of study. These advanced analyses required generation of acceleration-time histories that included the effects of SSI. The project structural engineer later used these acceleration-time histories as input motions for non-linear dynamic finite element analyses of the building.

SEISMIC SOIL STRUCTURE INTERACTION ANALYSES

Methodology

The SSI effects were conducted using the two-dimensional finite element (FE) program FLUSH (Lysmer et al. 1975). FLUSH considers the variation of ground motion and dynamic soil properties with depth, and the non-linear and energy-absorbing characteristics of the soil. The analysis is performed in the frequency domain, and soil non-linearity is modeled using the equivalent-linear method. In this method, a dynamic shear strain was assumed and dynamic soil properties are calculated for all elements, and a linear, frequency domain analysis is performed. The induced shear strain time histories are computed for each element and dynamic properties are adjusted accordingly. Using the revised dynamic soil properties, the analysis is repeated in an iterative manner until the computed soil properties are within 5 percent of the assumed values.

The structure was modeled in FLUSH using displacement-compatible isoperimetric quadrilateral elements (solid elements) and linear bending elements (beam elements). The beam elements were compatible at their nodes, where they have three degrees of freedom (two translational and one rotational.) A rigid base was attached to the lower boundary of the mesh to model the half-space. Transmitting boundaries were used along both the left and right hand side of the model to represent the dynamic stiffness of the semi-infinite layered system beyond the modeled area.

Element sizes for FE mesh were selected based on the energy transmission criteria:

$$h_{max} < (0.2)\lambda \qquad (1)$$

Where h_{max} is the maximum element height and λ is the wavelength corresponding to the highest frequency of the analysis. The wavelength, λ, is obtained by dividing the shear wave velocity by the highest frequency considered in the analysis ($\lambda = V_s/f_{max}$, where $f_{max} = 25$ Hz for these analyses).

The input to the analytical model consisted of a vertical or horizontal acceleration-time history. Both the horizontal and vertical ground motions were input as the control motion at the ground surface.

Cases Analyzed

The building was modeled in both its long and short directions. In each direction the belowground portion of the building was modeled using solid and beam elements and the aboveground portion was idealized using 11 lumped masses connected by beam elements to represent the mass and stiffness of the aboveground structure. Both finite element meshes included soil located 20 m laterally from the basement walls, and 8 m below the building's mat foundation.

The short direction finite element mesh included the entire belowground portion of the building (Figure 2). The finite element mesh in the long direction took advantage of building's symmetry in this direction by modeling only half of the basement structure (Figure 3).

Each FE mesh was analyzed using 6 horizontal ground motions (two horizontal components of 3 ground motions, corresponding to 2 risk levels). Recognizing that the building's response in the vertical direction would be same for both the short and long direction meshes, the vertical response was assessed using one finite element mesh for 6 ground motions.

Ground Motions

Seismic risk at the site is controlled by the Hayward Fault, which is located approximately 3 km east of the building. The analyses considered Hayward Fault design ground motions having a 10% probability of exceedance over 50-year and 100-year periods. The project engineering seismologist developed the surface motions using Joshua Tree, Lucerne, and Yermo seismograph station recordings of the 1992 Landers Earthquake, and altered these in the frequency domain to match the 10% in 50-year and

10% in 100-year target design response spectra. It is noted that the Joshua Tree, Lucerne, and Yermo seismograph stations were located behind, next to, and in front of the fault rupture. A total of 18 motions were developed for the analyses (3 components of 3 station recordings, for 2 seismic risk levels), each digitized to 4096 points at a constant time interval of 0.02 seconds.

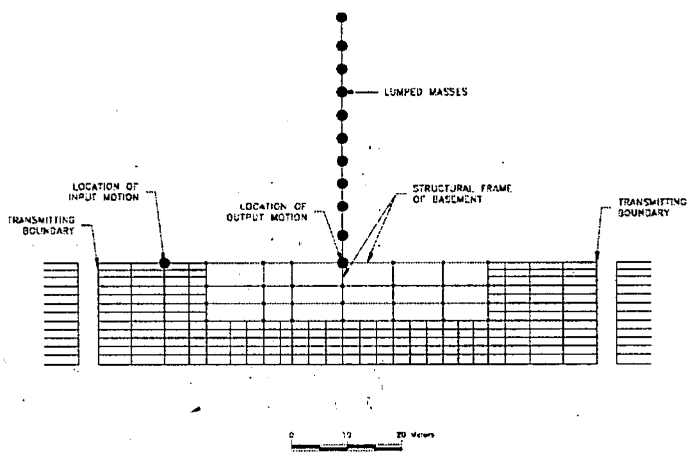

Figure 2. Short direction FLUSH finite element model.

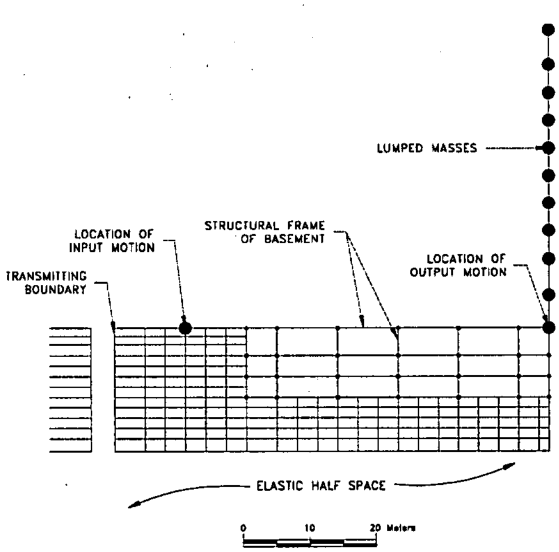

Figure 3. Long direction FLUSH finite element model.

Dynamic Properties of Soil

Geotechnical test borings indicated the area near the building was underlain by about 4 feet of silty sand fill over approximately 40 feet of medium-dense to dense Merritt sands of the San Antonio Formation. The Merritt sand is underlain by very dense silty and clayey sands and very stiff to hard silty clays of the Alameda Formation. Geophysical testing (Gibbs et al., 1977) conducted near the project site indicates that the shear wave velocity of the Merritt sand ranges from about 600 feet/second (fps) near the ground surface to about 1400 fps at depth. For these analyses, the shear wave velocity of the fill was estimated as 800 fps. The shear wave velocity of the Merritt sand was modeled to range from 800 fps at the fill-Merritt sand interface, to 1080 fps at a depth of 60 feet.

The stain-dependent shear modulus and damping of the fill soil and Merritt sand were modeled using the upper and lower bound (respectively) dynamic soil property curves developed by Seed and Idriss (1970).

Structural Properties of Building

The basement walls were considered as shear walls and interior columns were modeled as beams with fixed ends. The wall and column thickness varied from 30 cm to 36 cm. The basement walls and columns contribute to the stiffness of the basement structure in both the longitudinal and transverse directions. Thus, the stiffness of the basement structural members in the two-dimensional FLUSH model was adjusted to capture the contribution of the two short-direction shear walls in the long direction mesh; and the two long-direction shear walls in the short direction.

The stiffness, K, was defined as:

$$K = P/\Delta_{tot} \tag{2}$$

Δ_{tot} is the total displacement due to shear and bending when a unit load P is applied to the structure. For fixed walls, deflection due to shear and bending was computed as:

$$\Delta_{tot} = \Delta_{shear} + \Delta_{bending} \tag{3}$$
$$\Delta_{shear} = (1.2\ Ph)/(AG) \tag{4}$$
$$\Delta_{bending} = (Ph^3)/(12EI) \tag{5}$$

Where:
P = Lateral load
h = Height of wall
E = Elastic modulus
G = Shear modulus
I = Moment of inertia
A = Cross sectional area

Because the FLUSH model is two-dimensional, the stiffness of the structure presented in the model as a unit width was adjusted to account for the actual three-dimensional stiffness. This was accomplished by obtaining the stiffness of the belowground portion of the structure from a three-dimensional model of the structure and dividing the results by the width of the building perpendicular to the direction of analysis. The stiffness of structural elements representing the basement (perimeter walls, beams and columns) was then adjusted to result in the same calculated stiffness for the unit width.

The aboveground portion of the structure was modeled as a "stick" model with beams and lumped masses at each level. The stiffness of the beams was computed to produce the estimated natural frequency of the actual structure.

RESULTS

Table 1 presents the results of the analyses in terms of maximum acceleration for corresponding pairs of free field (FLUSH input motion) and structure (output) motions. The output motions represent shaking at the ground level, center of the building (reference Figures 2 and 3). The results are shown as response spectra ratios ($Sa_{structure}/Sa_{free\ field}$) for horizontal and vertical motions in Figures 4 and 5.

The results indicate that SSI slightly increases both the horizontal and vertical maximum acceleration in the building. Exceptions are noted for the Joshua Tree vertical motions, which show a slight decrease in maximum acceleration, thereby suggesting some beneficial effects of SSI. It is noted that these results are within the accuracy of the analysis and the range of data presented by Stewart (1999) and Poland et al. (1993) for studies of SSI effects on similar types of buildings.

Table 1 – Maximum acceleration for corresponding pairs of free field and structure motions

Direction - Risk Level	Joshua Tree Ground Motion	Lucerne Ground Motion	Yermo Ground Motion
Short - 10% in 50 yrs.	$A_{max (F.F)} = 0.64$ $A_{max (Build)} = 0.66$ (4% difference)	$A_{max (F.F)} = 0.66$ $A_{max (Build)} = 0.72$ (4% difference)	$A_{max (F.F)} = 0.64$ $A_{max (Build)} = 0.67$ (5% difference)
Short - 10% in 100yrs.	$A_{max (F.F)} = 0.79$ $A_{max (Build)} = 0.82$ (4% difference)	$A_{max (F.F)} = 0.79$ $A_{max (Build)} = 0.86$ (9% difference)	$A_{max (F.F)} = 0.77$ $A_{max (Build)} = 0.81$ (5% difference)
Long - 10% in 50 yrs.	$A_{max (F.F)} = 0.62$ $A_{max (Build)} = 0.66$ (5% difference)	$A_{max (F.F)} = 0.66$ $A_{max (Build)} = 0.68$ (3% difference)	$A_{max (F.F)} = 0.63$ $A_{max (Build)} = 0.66$ (6% difference)
Long - 10% in 100 yrs.	$A_{max (F.F)} = 0.76$ $A_{max (Build)} = 0.81$ (6% difference)	$A_{max (F.F)} = 0.81$ $A_{max (Build)} = 0.84$ (4% difference)	$A_{max (F.F)} = 0.77$ $A_{max (Build)} = 0.82$ (6% difference)
Vertical - 10% in 50 yrs.	$A_{max (F.F)} = 0.64$ $A_{max (Build)} = 0.63$ (-1% difference)	$A_{max (F.F)} = 0.73$ $A_{max (Build)} = 0.74$ (1% difference)	$A_{max (F.F)} = 0.68$ $A_{max (Build)} = 0.69$ (1% difference)
Vertical - 10% - 100 yrs.	$A_{max (F.F)} = 0.85$ $A_{max (Build)} = 0.84$ (-1% difference)	$A_{max (F.F)} = 0.96$ $A_{max (Build)} = 0.96$ (no difference)	$A_{max (F.F)} = 0.90$ $A_{max (Build)} = 0.91$ (1% difference)

Figure 4 shows that the intensity of the horizontal ground motions are up to 30 percent higher in the building compared to the free field over the period range of 0.02 sec to 0.25 sec. These differences are most pronounced for the Lucerne input motion, which contained slightly more high frequency energy than the Joshua Tree and Yermo ground motions. There is, for practical purposes, no difference between the free field ground motion and the ground-level motion in the structure at periods in excess of 0.25 seconds. As noted earlier, the finite element meshes were developed to have an energy transmission criterion of 25 Hz, and thus the high frequency ($T < 0.04$ sec) data presented in Figure 4 may be unreliable.

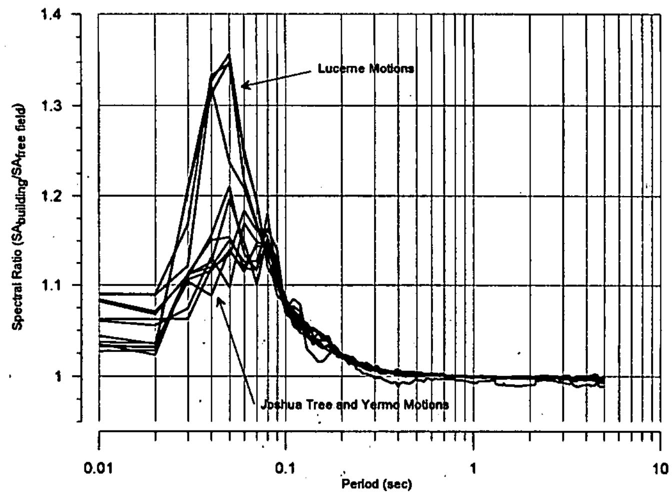

Figure 4. Response spectra ratios ($SA_{structure}/SA_{free field}$) for horizontal motions (based on response spectra damping of 5%).

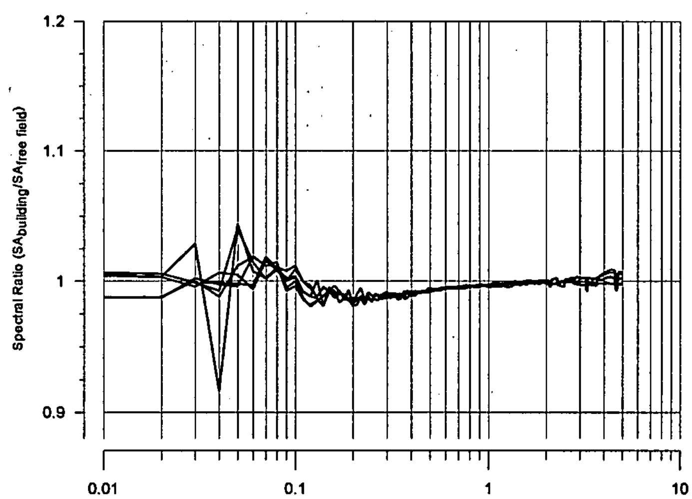

Figure 5. Response spectra ratios ($SA_{structure}/SA_{free field}$) for vertical motions (based on response spectra damping of 5%).

The response spectral ratios shown in Figure 5 indicate that vertical free field ground motion is virtually identical to that computed in the structure at all periods. Once again, the energy transmission criterion limits the reliability of the data at higher frequencies ($T < 0.04$ sec).

It is noted that parametric studies performed as part of the quality assurance review of these analyses indicated that the unfavorable effects of SSI lessened with decreased stiffness of the soils surrounding the building. Thus, the results presented here would have been more favorable had softer materials such as soft clays or loose sands surrounded the building. The parametric analyses also indicated that SSI effects were moderately sensitive to the structural properties of the aboveground portion of the building, and generally insensitive to the structural characteristics of the basement levels.

CONCLUSIONS

The results of these studies indicate that SSI has a negligible effect on horizontal ground motions for periods more than 0.25 seconds and at and near the building's predominant period (T ~ 1.8 seconds). Unfavorable SSI effects were found at higher frequencies and this could have some effect on mechanical or computer equipment with high natural frequency housed in the building. The effects of SSI on the building vertical motions in the building were for practical purposes, negligible.

Parametric studies indicate that more favorable SSI effects may have been realized if the building was surrounded by softer soils.

ACKNOWLEDMENTS

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