STRUCTURAL POUNDING OF ADJACENT MULTI-STOREY STRUCTURES CONSIDERING SOIL FLEXIBILITY EFFECTS

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SUMMARY

Collisions between adjacent structures due to insufficient separation gaps have been witnessed in almost every major earthquake since the 1960's. Although many analytical, numerical and experimental studies have been conducted into the pounding phenomenon, the number of those which take into account the effects of the underlying soil flexibility on the dynamic response of the pounding structures are almost negligible. The present study incorporates the effects of soil flexibility on the inelastic dynamic response of a specific structural configuration comprising adjacent 12- and 6-storey reinforced concrete moment-resisting frames. The time-history response of this system (linear soil behaviour and nonlinear structural response) subjected to an actual earthquake record is evaluated by means of the structural analysis software RUAUMOKO developed at the University of Canterbury. The total duration of the dynamic time-history analysis is twenty seconds, which comprises the first fifteen seconds of the 1940 El Centro earthquake (N-S component) applied at the base of the soil-structure interaction models and a five-second free-vibration phase. Impacts at storey levels only are considered.

This study evaluates the degree of approximation inherent in studies which neglect the effects of soil flexibility. The pounding response of the adjacent structures is found to be highly sensitive to the characteristics and direction of the seismic excitation.

INTRODUCTION

Structural pounding is mainly attributed to the difference in the dynamic properties of adjacent structures. The disparities in mass, stiffness, and/or strength result in out-of-phase lateral displacements under external excitations. Impacts will occur if these out-of-phase displacements exceed the available separation gap between the structures. The magnitudes of the impact forces and the locations of impacts along the heights of the structures depend on the magnitude of the existing separation gap, the extent of the disparity between the dynamic properties of the impacting structures, and the characteristics of the excitation. It is therefore apparent that, under certain conditions, the properties of the supporting soil must also be taken into consideration due to its influence on the above aspects.

The purpose of the present study is the investigation of the influence of soil flexibility on the dynamic response of a particular case of adjacent moment-resistant reinforced concrete frame structures subjected to pounding. The term "soil flexibility" used herein includes the effect of the soil properties on both the dynamic response characteristics of each of the individual oscillating systems and the through-soil interaction between the adjacent structures. A structural analysis software package developed at the University of Canterbury (RUAUMOKO, [Carr, 1998]) is utilised to conduct the inelastic time-history analyses. A Newmark Constant Average Acceleration integration scheme ($\beta = 0.25$) is implemented in the dynamic time-history analyses with a time-step of 0.001 seconds to capture the short-duration impact events. The capabilities of RUAUMOKO have been investigated in the experimental study of Filiatrault and Wagner, 1995.

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Impacts at storey levels only are considered. The first fifteen seconds of the N-S component of the 1940 El Centro earthquake (peak ground acceleration PGA = 0.348g) is utilised as the external dynamic excitation. In addition to the fifteen-second forced-vibration phase, a free-vibration phase of five-second duration is allowed at the termination of the seismic excitation. The effect of seismic attack from opposite directions is also investigated.

REVIEW OF OBSERVED CASES OF STRUCTURAL POUNDING

The first mention of structural pounding in the literature may have been as early as 1926 [Ford, 1926] in which the pounding of non-structural components against the structural elements of a building was discussed and the provision of a sufficient separation gap and proper detailing were recommended. Since then, the increase in urban development and the associated increase in real-estate values has compelled developers and designers to maximise land usage.

Although the Mexico City earthquake of 1985 is often cited as the most important single event in which extensive pounding damage was reported [Rosenblueth and Meli, 1986], the actual severity of the damage attributed directly to pounding may have been overstated [Anagnostopoulos, 1996]. Nonetheless, the potential structural and non-structural damage due to pounding should be assessed, be it during the design stage or in the seismic assessment of structures. Ample provisions should be implemented to minimise the potential threat to human life (such as may be caused by falling debris, e.g. glass or concrete, or the loss of a structural element, e.g. failure of a column due to sustained pounding at its mid-height) and to limit the resulting financial losses which may be incurred by the owner(s).

Cases of structural pounding have been reported in more recent earthquakes such as the 1994 Northridge earthquake [Tsai, 1997] (pounding of base-isolated buildings against their stops), the 1995 Hyogo-ken-Nanbu (Kobe) earthquake of 1995 [Park et al., 1995] (collision of pedestrian bridges between buildings) and the 1998 Colombia earthquake (Restrepo, per.comm.).

PREVIOUS STUDIES INCORPORATING SOIL-STRUCTURE INTERACTION

Although extensive efforts have been expended towards investigation of most aspects of the pounding phenomenon, the influence of soil conditions on the pounding response has been neglected. In order to facilitate the analytical and numerical representation and solution of the equations pertinent to the pounding phenomenon, conventional studies assume a fixed-base connection between the impacting structures and the foundation soils upon which they rest in addition to neglecting the through-soil interaction between these structures. The former assumption implies a very stiff foundation soil. This may be true in the case of low structure mass to soil stiffness ratio but is not valid for cases of heavy structures constructed on soft soils. The neglect of through-soil interaction implies the complete isolation of the oscillating structures from the soil environment in which they are constructed.

To the authors' knowledge, the only study that specifically investigates the influence of soil-structure interaction on the pounding response is the analytical investigation conducted by Schmid and Chouw, 1992. A boundary-element solution was applied in the modelling of the continuous half-space (soil mass) underlying the two buildings. While through-soil coupling was not considered, they concluded that soil-structure interaction effects had a profound influence on post-impact behaviour which cannot be extrapolated from an analysis assuming fixed-base conditions.

Anagnostopoulos and Spiliopoulos, 1992 modelled the supporting soil mass as a discrete element (spring-dashpot) system for the translational and rocking degrees-of-freedom. A five-fold reduction in the soil stiffness resulted in an increase in the natural periods of the oscillating systems and hence to a reduction of the displacement ductility demands of the impacting inelastic multi-degree-of-freedom systems and to a reduction of storey shears in the elastic analysis.

Kasai et al., 1992, in their survey of structural pounding damage which occurred during the 1989 Loma Prieta earthquake, noted the correlation between the incidence of pounding and the soil conditions prevalent at these sites. They postulated that the increased intensity of shaking due to soft soil conditions and/or the possible occurrence of structural settlement and rocking at these sites might have been contributing factors to pounding.

Valles and Reinhorn, 1997 suggested a mathematical expression to determine the critical separation gap required to preclude pounding. In this equation, an expression accounting for foundation rotation due to soil-structure interaction effects was included.

EFFECTS OF POUNDING ON DYNAMIC RESPONSE

Impacts due to structural pounding transmit short duration, high amplitude forces to the impacting structures and may occur at any level of the colliding structures and at any location along the impacting levels (in the case of disparate storey heights of the adjacent buildings resulting in slab-column impacts). The detrimental effects of these forces may be enumerated as follows:

- High amplitude, short duration local accelerations which are generally not accounted for in design.

- Localised degradation of stiffness, and/or strength in impacting members. In addition to the adverse effects this will entail on the strength of the members, the distribution of shear and flexural forces will also be affected. Since the design of earthquake resistant structures in most codes is based upon the capacity design method [Paulay and Priestley, 1992] this is an especially significant factor which must be accounted for when assuming plastic hinge locations.

- Modification to the overall dynamic response of structures. Either an amplification or de-amplification of response will be sustained by structures, depending on the relative dynamic characteristics (mass and stiffness) of the impacting structures on one hand and the supporting medium on the other, in addition to the characteristics of the seismic excitation. Modes of response not accounted for in design may be introduced, for example torsion.

SOIL-STRUCTURE INTERACTION

The effects of consideration of soil-structure interaction in a dynamic analysis are twofold. The first effect is on the free-field excitation and the second is on the response characteristics of the oscillating system, as outlined below.

The presence of a soil layer above the bedrock will lead to a reduction in the bedrock motion due to overburden pressure. The ground motion at the free-field will either be amplified or attenuated depending on the characteristics of the ground motion and site conditions.

The main effect of soil-structure interaction on structural response is the introduction of a rocking vibration mode and the increase of the natural periods. In addition, system damping will increase due to propagation of waves from the foundation-soil interface (radiation or geometric damping) and material (hysteretic) damping in the soil. The fundamental frequency of the soil layer determines the extent of radiation damping which will occur. If the excitation frequency is less than the fundamental frequency of the soil layer (as in the case of a shallow layer underlain by bedrock), damping will not occur and only material damping is present. This property of the soil layer is known as the cutoff frequency [Wolf, 1994].

In this study, the material damping of the soil was not accounted for in the soil-structure interaction models and only radiation damping was represented. This was due to the application of a constant damping model in the numerical model of the total structure-soil system. This model was deemed to be representative of the total system damping. In addition, at the time this study was conducted RUAUMOKO did not allow for the specification of the individual damping characteristics of the various components (moment-resistant frames, impact elements and soil mass) which comprise the total system. Subsequent software upgrades, however, have overcome this limitation.

NUMERICAL MODELLING

Modelling of moment-resistant frames

The configuration incorporated in this study comprised of adjacent two-bay twelve- and six-storey reinforced concrete moment-resistant frames of equal storey heights (3.65m) designed according to the principles of the capacity-design method [Paulay and Priestley, 1992]. The twelve-storey two-bay Rahman frame [Rahman, 1999] is designed in accordance with the latest versions of the relevant New Zealand design codes [NZS 4203, 1992, NZS 3101, 1995] while the six-storey two-bay Jury frame [Jury, 1978] was designed in compliance with earlier versions of the same codes. The mass ratio of the two buildings is 1:2.28 while the ratio of fixed-base periods is 1:2.09. The more flexible Rahman frame is located to the left of the Jury frame. Plastic hinge zones, rigid end blocks and reduced effective areas of structural elements due to inelastic behaviour were all accounted for in the numerical modelling.

Travelling wave effects in the floor diaphragm, in addition to its relative flexibility, lead to the gradual contribution of the nodal masses at the same level at subsequent time stations. This is accounted for in the models of the present study through the independence of the lateral degrees of freedom of the nodes of each frame (i.e. the horizontal degrees of freedom for each level are unslaved).

Numerical modelling of the floor-to-floor impacts is achieved through the utilisation of a CONTACT-type member with a Hertz contact rule. The non-linear nature of this rule allows the representation of the increasing contact area which is expected during a pounding event. The impact element stiffness is based on the axial stiffness of the beams of the 12-storey frame to which is added an arbitrary width of slab [NZS 3101, 1995].

Modelling of soil flexibility

A discrete element representation, comprising a mass-spring-dashpot system for each degree-of freedom, was implemented in this study. This system is representative of the dynamic properties of the underlying elastic, homogeneous soil medium. The discrete-element model of Mulliken and Karabalis, 1998, utilises the coefficients derived by previous investigators ([Richart et al., 1970], [Gazetas, 1983] and [Wolf, 1994]) in the development of a discrete element model. These coefficients are functions of the geometric properties of the foundations and the characteristics of the supporting soil medium (Poisson's ratio, mass density and shear wave velocity). In addition, a spring-dashpot element simulating the coupling of vibration modes between the adjacent foundations is also applied. The coefficients of this spring-dashpot system were derived through the application of suitable curve-fitting formulae to obtain a best-fit with an existing rigorous boundary element solution. While the Karabalis and Mulliken model allows for travelling wave effects in the coupling coefficients, this feature was not utilised in the present study due to the small separation gaps investigated. A schematic representation of the numerical model implemented in the numerical analyses is shown in Figure 1.

A clay soil was assumed with the a shear modulus (G) of 18 750 kN/m^2, a Poisson ratio (ν) of 0.5. The mass density (ρ) is assumed equal to 1800 kg/m^3 from which the shear wave velocity (v_s) is determined to be 102.062 m/s.

RESULTS AND OBSERVATIONS

For the particular structural configuration investigated, the pounding forces at level 6 were the largest for all separation gaps and remained so for all conditions of foundation fixity. Therefore, the influence of base fixity conditions (fixed base = FB, compliant with no coupling = NC, and compliant with through-soil interaction = FC) on impact force magnitudes and impact-side nodal accelerations for only the 1mm separation gap case is presented below. In addition, the influence of direction of the seismic excitation (1940 El Centro earthquake) is also discussed.

The effect of foundation compliance is to increase of the natural periods of both structures from the fixed base condition (Table 1). Consideration of the flexibility of the soil mass between the structures, even for the large separation distance considered herein (2 metres), shows a slight difference from the compliant foundation case. Inspection of the displacement time-history for level 12 of the Rahman frame (Figure 2) reveals that the increase in natural period, while having only a small influence on maximum displacements, is more prominently exhibited in the free-vibration segment of the time-history.

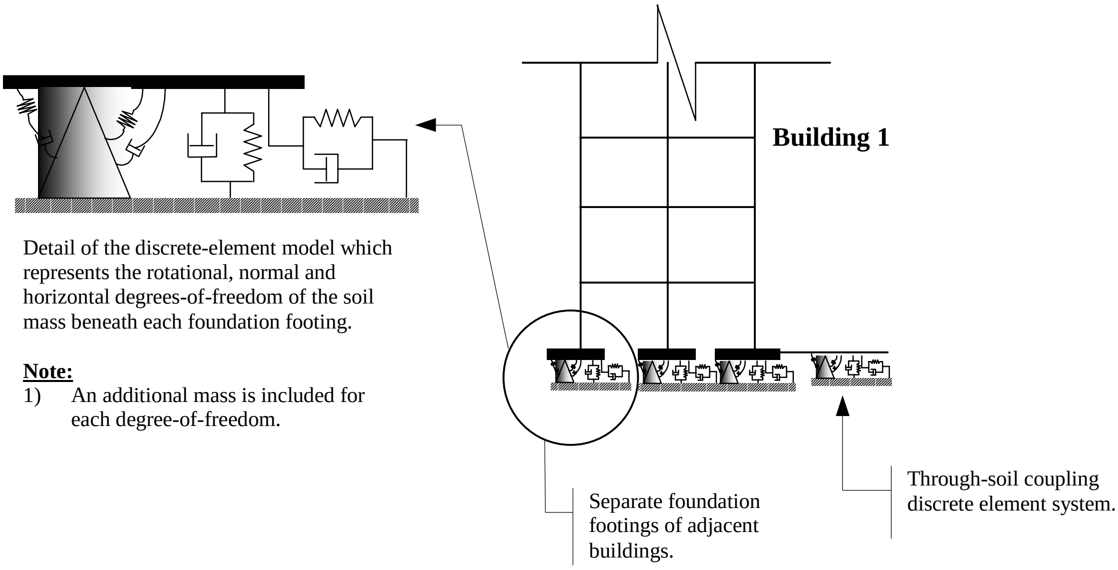

Figure 1: Numerical model representative of the moment-resisting frame and soil flexibility.

The influence of the foundation fixity assumption and direction of seismic attack on the magnitudes of the maximum impact forces at the six levels of impact are shown in Figure 3. The results are more sensitive to the foundation conditions for the case of seismic attack from the direction of the 12-storey building.

The accelerations developed in the impact-side nodes are presented in Figures 4 and 5 for the positive and negative excitations, respectively. These response values are more sensitive to the foundation conditions than the maximum impact forces. The increases in the maximum nodal accelerations do not correspond to the increases in maximum impact forces due to foundation compliance. For example, for the positive direction excitation (Figure 4) the acceleration at level 5 in the Rahman (12-storey) frame is higher than that at level 6 despite the larger maximum impact force at the latter storey. This is due to maximum impacts occuring at the same time (3.5 seconds) at levels 4 to 6 which corresponds to a peak in the seismic excitation. While the impacts at levels 3 to 5 occurred at the same time (approximately 4 seconds) for the opposite direction of excitation, these impacts corresponded to a very small excitation pulse in the direction opposite to the impacts. The large (2.25g) accelerations at level 6 of the Rahman frame (Figure 5) are due to an impact occurring at the same time as a peak in the excitation.

Table 1: Effect of foundation fixity on natural period of 12- and 6-storey moment-resisting frames.

	Natural Period (seconds)	
Foundation fixity	**12-storey (Rahman) frame**	**6-storey (Jury) frame**
Fixed	2.272	1.086
Compliant	2.427	1.139
Coupled	2.406	1.136

CONCLUSIONS

Confidence in the assumption of fixed-base foundation conditions in pounding studies is not justified as the results are highly sensitive to many parameters related to the structures and their numerical modelling in addition to the prevalent soil conditions, the characteristics and direction of the expected seismic event. The influence of foundation fixity conditions on the nature of the free-vibration pounding time-history indicates the sensitivity of the results to the characterisitcs of the earthquake excitation. A period shift due to foundation compliance will alter the time-station at which first impact occurs with respect to excitation maxima, with consequences on subsequent impacts. Therefore, generalisations are not possible and each case must be investigated separately for the relevant configuration, site conditions, and expected seismic hazard. Further studies will be conducted investigating other separation gaps, structures of equal total heights, earthquakes of differing charactersitics and the influence of these parameters on other response values such as impact-side column shears.

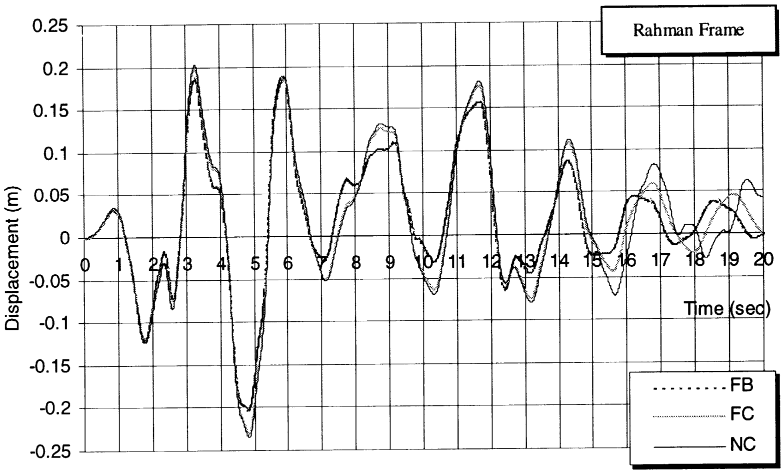

Figure 2: Effect of foundation fixity assumptions on maximum storey displacements of level 12 Rahman frame for no-pounding case.

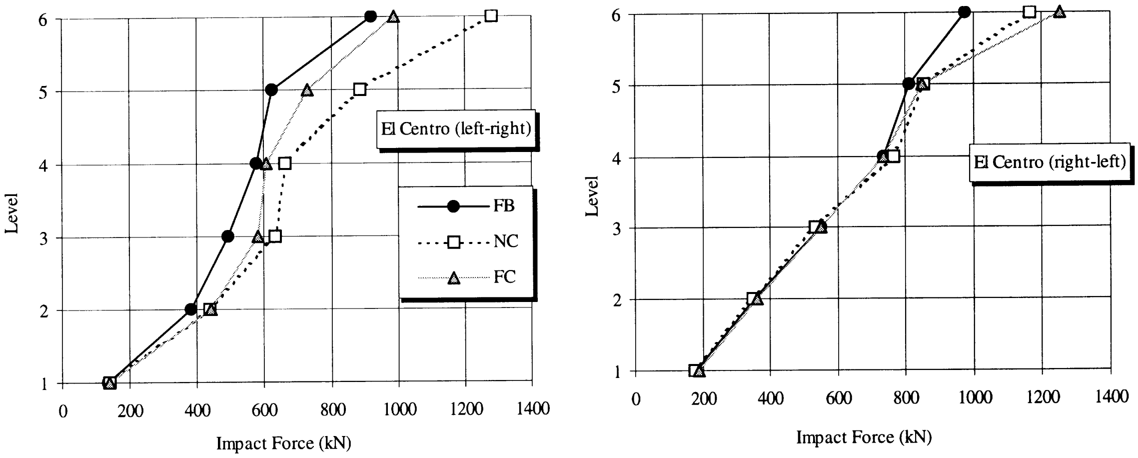

Figure 3: Effect of foundation fixity assumptions and direction of seismic attack on maximum impact forces for 1mm separation gap case.

N.B.: FB = Fixed Base, NC = Non-coupled (soil-structure interaction only), FC = Foundations coupled.

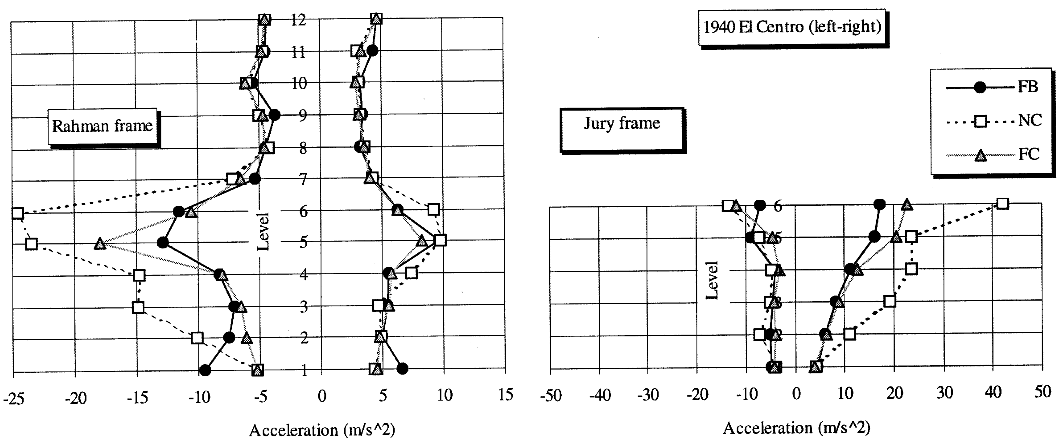

Figure 4: Effect of foundation fixity conditions on impact-side nodal accelerations for 1mm separation gap (El Centro applied left-right).

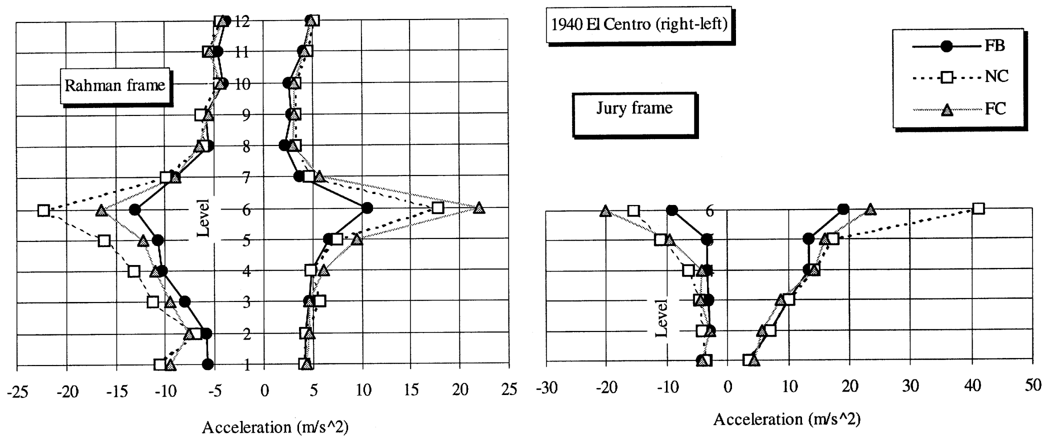

Figure 5: Effect of foundation fixity conditions on impact-side nodal accelerations for 1mm separation gap (El Centro applied right-left).

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