Modification of Free-Field Motions by Soil-Foundation-Structure Interaction for Shallow Foundations

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SUMMARY:
This paper describes the effects of soil-foundation-structure interaction (SFSI) on twenty two instrumented buildings in California. The buildings cover a wide range of shallow foundation configurations and site conditions and have records at both the foundation level and nearby free-field. A total of 99 earthquake records obtained from these building sites have been analysed. Comparisons of response spectra from the free-field with the corresponding spectra from the foundations clearly show the effect of SFSI. Sixty-six records showed significant reductions in spectral values at the foundation level for periods less than about 0.5- 0.6s and thirty-three records showed amplification in spectral values. Past procedures for analysing the effect of the foundation on the free-field input motions are all based on the assumptions that the foundation slabs always reduce the motion. ASCE41-06 recognizes that foundations with interconnected grade beams or concrete slab will always reduce the free-field motions except for buildings sitting on soft clay sites and having flexible roof and floor diaphragms. It also presents a formula for calculation of the spectral reduction factor for design. These methods and ASCE41-06 provisions were applied to the free-field data of those 66 records that showed reduction in spectral values to estimate the corresponding foundation motions. The results were then compared with the recorded data at the foundation level. From these comparisons it was observed that, in general, the agreement was poor. A preliminary study of the foundations, buildings and site conditions of 33 records showing spectral amplification found that these buildings had rigid floors diaphragms and were not on soft clay soils. Therefore according to ASCE41-06 the spectral response of the foundation should have been reduced instead of amplified. This discrepancy needs to be explored. More data on these buildings and sites is being sought to help clarify the discrepancy.

Keywords: Soil-structure interaction, Foundation modelling, Input motion, Base-slab averaging

1. INTRODUCTION

In conventional seismic design practice, it is assumed that base of the structure experiences the free-field ground motion. The structure is assumed to be fixed at the base and the analysis is carried out considering only target structure subjected to free-field input motion at the base with no regards of supporting soil. Until recently, most of codes were silent on the issue and some addressed the issue of soil-foundation-structure interaction (SFSI) but mostly limited to inertial effects that result into changes in fundamental period and damping of the system. The ATC-40(1996) and FEMA-356(2000) addressed the flexible foundation effects; Eurocode-8 (EC8, 2000) recognizes the kinematic effects but restrict to pile foundations. Recently, FEMA- 440 (ATC, 2005) and ASCE 41-06(ASCE, 2007) provide provisions on including SSI effects in nonlinear inelastic analysis. The basis of treatment of kinematic effect in these guideline and standard is based on work by Kim and Stewart (2003) who calibrated a model on spatial variation of wave proposed by Veletsos et al. (1997) on the basis of recordings of instrumented buildings. The calibrated model accounts the spatial variation of the incident waves and gives reduced seismic demand at the foundation level based on averaging the wave effect over the area of foundation base. While reduction in the motion at the base is observed in several buildings, there are observations in other buildings where foundation motions are amplified considerably increasing the seismic demand (Poland et al., 2000).

We studied twenty two instrumented buildings which have records both in foundation and nearby free-field site. In this study, we focused on the kinematic interaction between soil and stiff foundation slabs as this is regarded as the contributing factor to motion variation between free-field and foundation. Some observations are contrasted with state-of-practice analytical models suggested by other researchers.

1.1. Recorded Motions at Instrumented Buildings

Earthquake records were collected from sites of instrumented buildings in California using earthquake data base of CSMIP, PEER, USGS and COSMOS. Sites are selected such that the instrumented buildings should have records at the foundations and nearby free-fields at a distance of less than 500m. Table 1 provides list of buildings selected and details of the record including peak acceleration at each component. If there is more than one sensor available at foundation level, the sensor which corresponds with sensor at free-field most closely is chosen for comparison of motions.

The recorded motions are processed with baseline correction and filtering. Comparison between motions at free-field and foundation base are made in time histories and spectral values. Figure 1a shows 5% damping response spectra for acceleration, velocity and displacement of foundation and free-field records observed in San Bernardino 3-story office building in 1992 Lander Earthquake in N-S direction. Figure 1b is similar spectra for Hollister warehouse in 1989 Loma Prieta Earthquake in E-W direction. The PSDs and transfer functions between respective motions are illustrated in Figs. 1.2a and 1.2b.

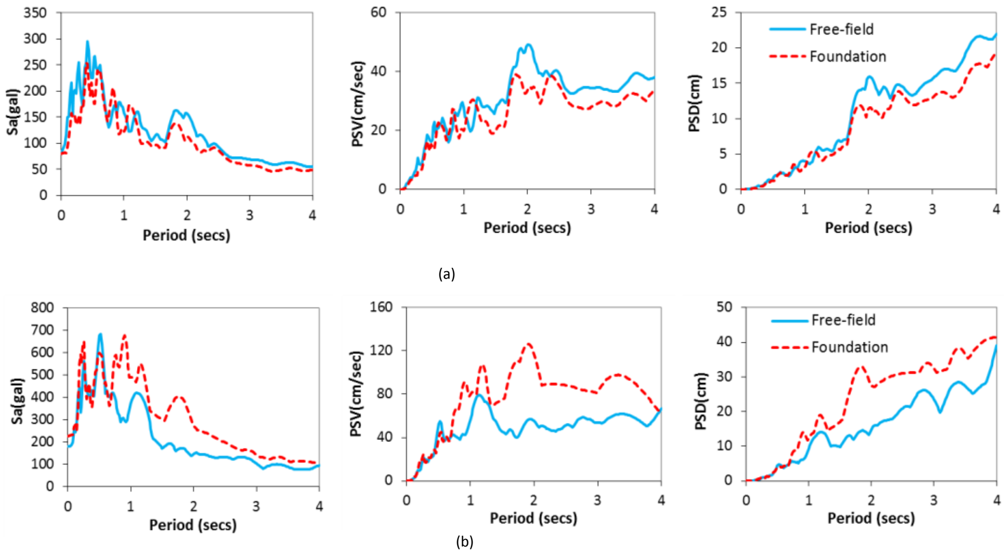


Figure 1.1. Spectral accelerations, velocities and displacements of motions at free-field and foundation at (a) San Bernardino 3-story office building in 1992 Landers Earthquake in north-south direction (b) Hollister Warehouse in 1989 Loma Prieta Earthquake in east-west direction.

In San Bernardino Office building, the motion is slightly reduced from free-field to foundation. Observe the reduction in PGA as well as in spectral acceleration, spectral velocity and spectral displacement over most of the period range. The difference is higher in short period range for spectral acceleration and in long period range for spectral velocity and spectral displacement. The two motions are mostly in phase. The motions at the foundation and free-field have peaks around same frequencies for significant part of the power spectral density (Fig. 1.1a) which suggest that there is no significant shift in period. The transfer function between foundation and free-field in the Fig. 1.2a shows that reduction in motion occurs for a frequency range higher than 6 Hz which coincides with very low power spectral density for foundation motion. This implies that the significance of the attenuation at the foundation is low in terms of its energy.

Table 1.1. Instrumented buildings selected for the study

No	Building	Earthquake(Epicentral distance in Km)	Free-field peak ground acceleration (gal)			Foundation level peak acceleration (gal)		
			EW	NS	Vert.	EW	NS	Vert.
1	RANCHO Cucamonga 4 st. Justice Cntr.	LD (106.0)	65.47	107.4	41.8	67.34	98.9	47.14
		NR(90.0)	38.33	63.16	31.1	37.4	36.26	25.7
		PS(90.1)	15.7	7.86	-	12.7	13.36	-
		UL(12.1)	214.18	248.24	4.13	104.56	119.48	4.03
		WT(47.0)	34.47	36.79	25.67	25.44	20.48	19.32
2	Pomona - 2-st. Commercial Bldg.	UL(9.9)	187.05	168.54	83.64	116.63	24.08	76.08
		WT(30.0)	42.96	52.17	-	42.02	47.81	-
3	Los Angeles - 7-st. Univ. Hospital	LD(163.0)	40.05	48.89	-	24.72	38.88	-
		NR(36.0)	222.36	477.91	15.45	160.04	379.03	82.35
4	Santa Cruz 5- st. Govt office Bldg.	GL(37.5)	27.44	33.48	21.17	14.6	21.12	7.84
5	Richmond 3-st. Govt Office Bldg.	LP(108.0)	132.42	109.36	32.97	97.63	128.56	31.62
6	Los Angeles - 14-st. Hollywood Storage	NR(23.0)	234.11	352.88	103.99	191.47	277.33	82.46
		WT(25.0)	-	200.29	-	-	114.4	-
7	Los Angeles - 15-st. Govt Office Bldg	LD(168.0)	30.85	30.63	16.75	30.69	29.28	14.08
		NR(32.0)	127.47	197.41	96.2	129.34	198.19	65.6
8	Seal beach 8 story Office Bldg.	LD(160.0)	43.93	40.82	17.11	45.59	37.33	14.48
		NR(66.0)	77.61	65.82	33.46	79.02	55.56	22.2
9	El Centro- 6-st Imperial Co. Services bldg.	IP (28.4)	38.62	234.50	233.88	342.06	294.79	150.16
10	Newport Beach - 11-st. Hospital	NR(86.0)	101.59	82.7	19.98	75.43	52.11	30.69
11	Long Beach - 7-st. Office Bldg.	NR(36.0)	65.02	55.01	23.39	65.58	44.44	18.05
12	San Bernardino 9-st. Comm Bldg.	LD(80.0)	90.78	83.86	57.77	76.95	80.58	38.36
13	San Bernardino 3-st. office Bldg	LD(80.0)	76.29	85.26	52.39	127.49	79.93	56.07
14	Los Angeles – 7-st. UCLA MathSc. Bldg .	NR(18.0)	47.77	431.96	269.42	226.84	280.05	198.5
15	Sylmar - 6-st. County Hospital	NR(16.0)	581.61	804.86	449.16	701.22	444.34	338.17
		WT(45.0)	51.54	55.5	38.077	54.38	52.29	46.57
16	Lancaster - 3-st. Office Bldg .	WT(70.0)	59.22	61.42	23.80	62.20	50.37	15.59
17	Los Angeles - 2-story Fire Command Cntr.	LD(161.0)	52.17	55.28	29.42	50.97	52.02	20.82
		NR(38.0)	257.98	331.27	131.94	209.94	166.78	109.85
		SM(28.0)	113.4	96.59	54.76	74.08	75.97	44.38
18	Park field - 1-st. School Bldg.	PF7(4.7)	16.27	13.98	-	11.54	14.03	-
19	Templeton 1-st Hospital	PF5(46.7)	10.35	18.57	3.95	9.66	10.75	4.05
20	King City - 2-st. Hospital	PF4(80.9)	47.44	39.44	47.55	28.45	30.72	30.09
21	Hollister - 1-story Warehouse	LP(48.0)	179.2	370.89	190.19	226.89	368.81	161.52
22	San Jose - 3-story Office Bldg	AR(24.5)	35.24	46.04	26.17	17.53	24.93	12.93
		LP(21.0)	-	286.65	209.61	-	154.92	115.76

LD: Landers Earthquake (1992); NR: Northridge Earthquake (1994); PS: Palmspring Earthquake (1986); UL: Upland Earthquake (1990); WT: Whittier earthquake (1987); GL: Gillory Earthquake (2002); IP: Imperial Valley Earthquake (1989); LP: LomaPrieta earthquake (1989); PF7: Parkfield Earthquake (2007); AR: Alumrock earthquake (2007); PF4: Parkfield Earthquake (2004); PF5: Parkfield Earthquake (2005) and SM:Sierramadre Earthquake (1991)

In Hollister Warehouse building, the foundation motion got amplified (Fig. 1.1b). Increase on motion is observed in PGA as well as in spectral quantities. The velocity and displacement spectra show larger difference between free-filed and foundation motions. The PSD and TF (Fig.1.2b) between motion at foundation and free-field show that the amplification is not limited to lower modes of vibration.

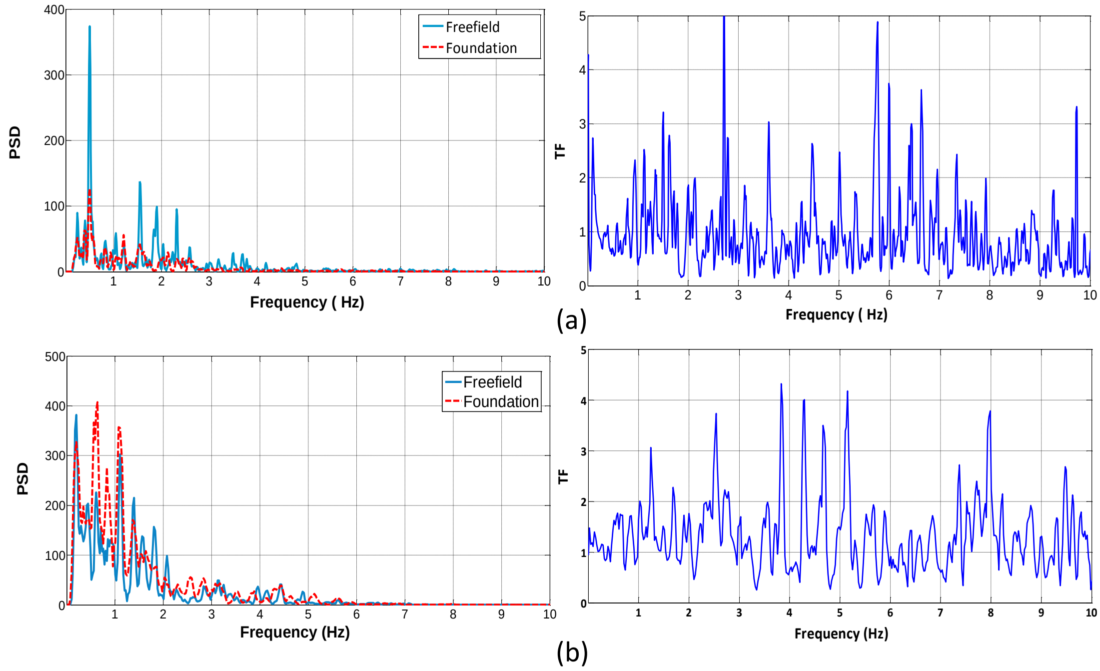


Figure 1.2. Power spectral density functions (PSD) of and transfer function (TF) between motions at free-field and foundation at (a) San Bernardino 3-story office building in 1992 Landers Earthquake in north-south direction (b) Hollister warehouse in 1989 LomaPrieta Earthquake in east-west direction.

2. REDUCTION AND AMPLIFICATION OF MOTIONS AT FOUNDATION LEVEL

Variations are observed to the foundation motion relative to the free-field motion in instrumented buildings with attenuation in most cases and amplification in some cases. Figure 2.1a shows the foundation motion in relation to free-field peak ground acceleration. While most of the pairs of the records show reduction, there are some buildings which clearly get motion amplified at their bases.

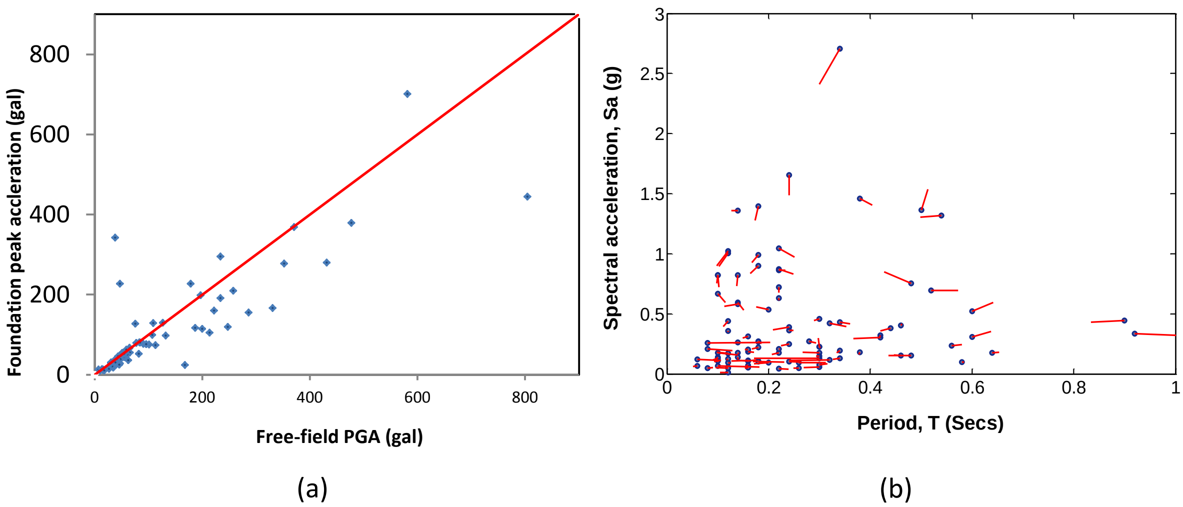
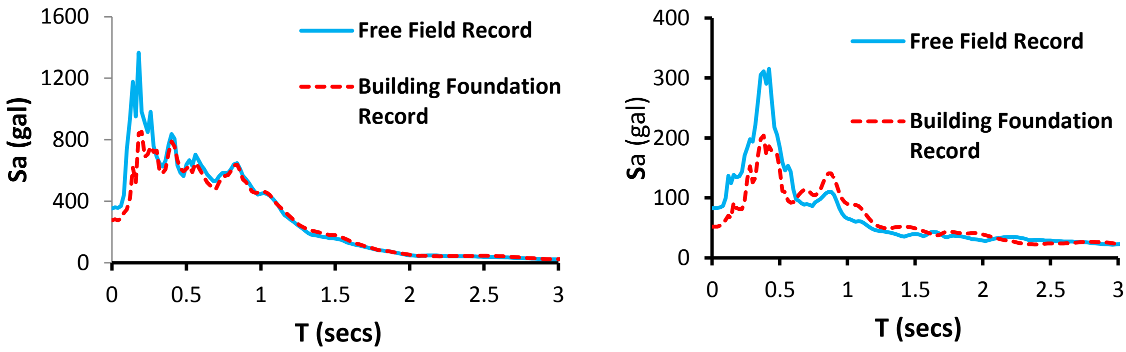


Figure 2.1. (a) Foundation peak acceleration in relation to free-field peak ground acceleration (b) Maximum spectral acceleration at free-field and corresponding shifts at building foundation

Figure 2.1b shows the variation in maximum spectral acceleration of the motion from free-field to building foundation. The circles plot the maximum spectral value at free-field at respective period and the line attached to them show the shift in time period maximum spectral value at the foundation. From the observed data in peak spectral acceleration does not provide clear trend in reduction or amplification.

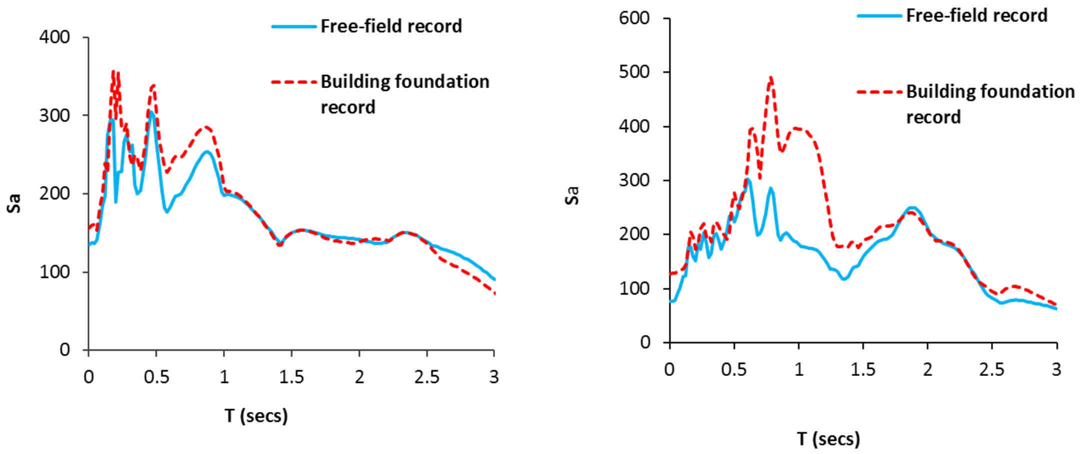
Figures 2.2a and 2.2b show reduction of free- field motion at the base of the structure. These response spectra are calculated from the time history acceleration records at the base of instrumented buildings and corresponding free-field motions recorded under the California Strong Motion Instrumentation Program (CSMIP). The reduction of motion at the foundation base is more dominant in the short period range. Out of 99 pairs of records of instrumented buildings with foundation with rigid slab or interconnected strip footing resting on ground (shallow foundations with no basement), two-thirds of the records in buildings with surface foundations showed a reduction in motion at the foundation level compared corresponding free-field.



(a) (b)

Figure 2.2. Reduction of motion at the foundation level compared to free-field at (a) Hollywood Storage Building in Northridge Earthquake (N-S) (b) Newport Beach office building in Northridge Earthquake (N-S)

Figures 2.3a and 2.3b show amplification of motion at the foundation level compared to free-field.



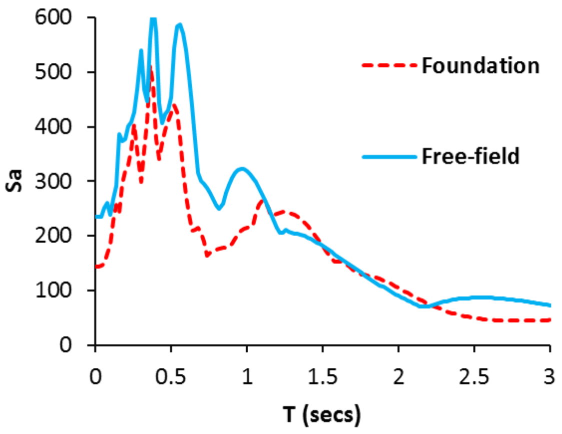
(a) (b)

Figure 2.3. Amplification of motion at the foundation level compared to free-field at (a) Fortuna Supermarket in 1992 Petrolia Earthquake (N-S)(b) San Bernardino building in Landers Earthquake (E-W)

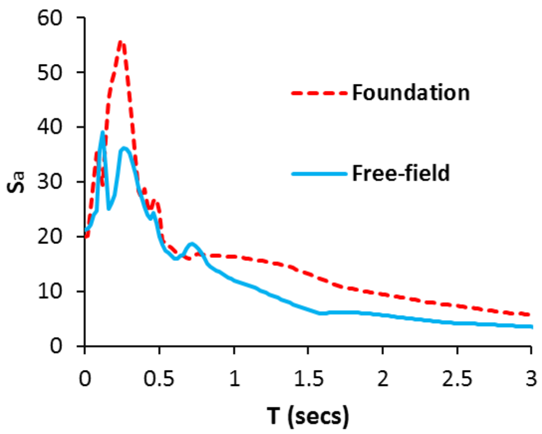
Out of 99 pairs of records of instrumented buildings with foundation with rigid slab or interconnected strip footing resting on ground (shallow foundations with no basement), one-thirds of the records in buildings with surface foundations showed amplifications in peak acceleration or spectral accelerations. The period range where the motion get amplification or de-amplification is large and do not necessarily limit within the vicinity of the fundamental period of structures.

2.1 Same Building Site with Different Response for Different Earthquakes

In some of the building sites, it is observed that foundation motion got reduced compared to free-field in one earthquake but got amplified in another earthquake. Figure 2.4 (a) shows the comparison of foundation motion at 5-story Eureka residential building with corresponding free-field spectra in 2010 Fernadale Earthquake in N-S direction. There is significant reduction at foundation motion. However, the same building was subjected to Trinidad Earthquake in 2008 when amplification of foundation motion was observed (Fig. 2.4b). This shows that the characteristics of the excitation affect the response at the foundation level.



(a)



(b)

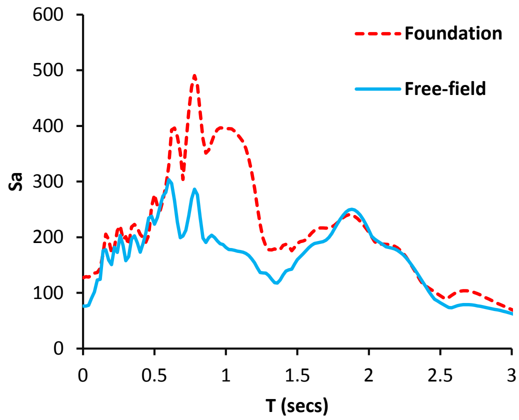
Figure 2.4. Spectral motion at the foundation level compared to free-field at Eureka 5-story residential building in N-S direction in (a) 2010 Ferndale Earthquake (b) 2008 Trinidad Earthquake

2.2 Same Building Site with Different Response to Same Earthquake in Two Directions

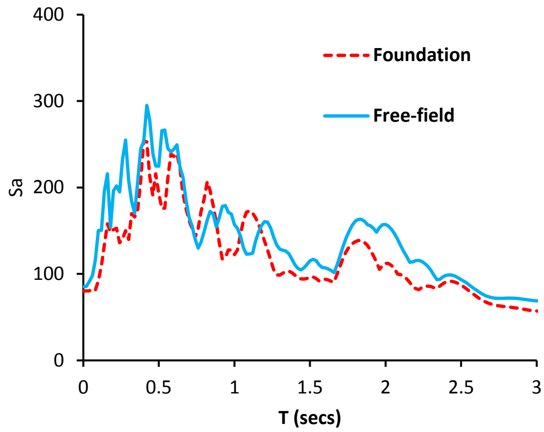
In some of the buildings, it is observed that same earthquake can produce different response in longitudinal and transverse direction. Figure 2.5a shows the comparison of foundation motion at 3-story san Bernardino office building with corresponding free-field spectra in 1992 Lander earthquake in E-W direction. The foundation motion was amplified in the period range of 0.5-1.5 secs. However in the N-S direction, motion was reduced at the foundation over entire period range. The foundation plan is rectangle with comparable dimension of 135ft by 120ft. The difference in motion for same earthquake in two directions could be attributed to different properties of the building in two directions. The characteristics of motion those directions may also play the role.

2.3 Consistency in the Response

Many building sites, however, show a consistent behaviour in terms of motion variation from free-field to foundation level in different earthquakes under different direction. Records at Newport Beach building shows that foundation motion is always reduced in both direction of the building. El Centro Imperial Co. service building showed amplification in both directions to the earthquakes.



(a)



(b)

Figure 2.5. Spectral motions at the foundation level compared to free-field at San Bernardino 3-story office building in 1992 Landers Earthquake in (a) E-W direction (b) N-S direction

3. FEMA-440 PROVISION FOR KINEMATIC EFFECT

ASCE41-06 (ASCE, 2007) has provided reduction factors for spectral values due to the action of foundation slab as shown in Fig. 3.1 for slab foundation with shallow embedment. The foundation input motion for subsequent building analysis and design is obtained by applying the period dependent reduction factor to the code spectrum. Based on charts of reduction values developed for effective width ranging from 65ft to 330 ft, following equation is provided to calculate the reduction in response spectra for base slab averaging (RRS_{bsa})

$$RRS_{bsa} = 1 - \frac{1}{14{,}100}\left(\frac{b_e}{T}\right)^{1.2} \quad \geq \text{ the value for } T = 0.2\text{sec} \tag{3.1}$$

Where $b_e = \sqrt{(a.b)}$ is effective foundation size in feet, a and b are longitudinal and transverse dimensions of footprint of building foundation in feet and T is fundamental period of building in sec.

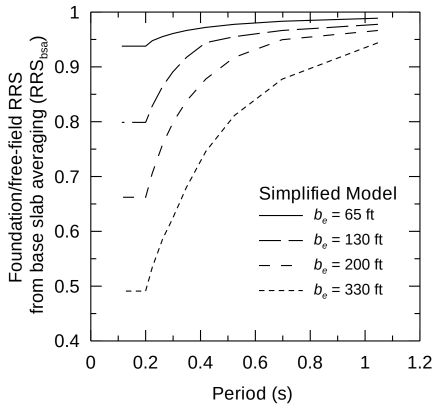


Figure 3.1. Ratio of Response spectra for base slab averaging, RRS_{bsa} (ASCE, 2007)

The basis of the ASCE provision is kinematic interaction effect where foundations get reduced motion compared to free-field based on work by Kim and Stewart (2003) that suggests the reduction of motion at foundation level of buildings with foundation of concrete slab or interconnected components with grade beam. The reduction is not allowed for buildings located in soft clay site and consisted of flexible floor and roof diaphragms.

3.1. Spatial Incoherency Model

The simplified model presented in ASCE41-06 is derived based on spatial variation model proposed by Veletsos et al. (1997) for base slab averaging. The transfer function in the model is to be less than unity implying the reduction of foundation motion compared to that of free-field. Incoherency parameter used in the model was later calibrated by Kim and Stewart (2003) based on data of seismic records in instrumented buildings with shallow foundations. They developed a simple procedure of estimating the transfer function from only input of foundation dimension.

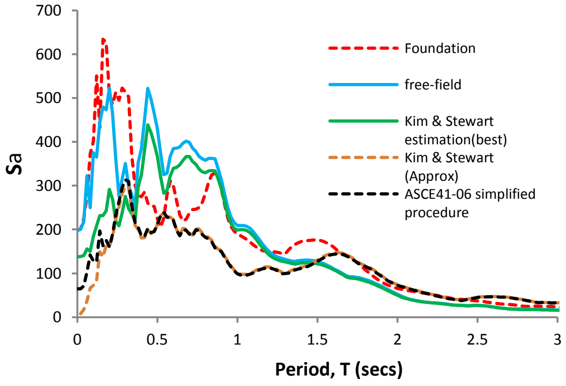
The approach of this procedure is based on assumption that variation of ground motion between two points can be fully characterized by the spatial incoherency reflected by single parameter, *k*. In the calibration, it is assumed that the variation of motion from free-field to foundation can be entirely attributed to the kinematic effect through spatial incoherency. However, the dynamic effect of building mass and mass at the foundation level including surrounding soil can contribute to the rise amplification or reduction of the motion depending on dynamic characteristics of building as well as soil mass.

3.2. ASCE Simplified Procedure

The transfer function from model described in earlier section applies to the frequency domain. The time history of ground motion needs to be transformed into frequency domain. The frequency amplitude should be multiplied with transfer amplitude. The product is then needs to be transfer back to time history by performing inverse Fourier transformation. Kim and Stewart (2003) suggested that the amplitude can be directly applied to design response spectra for the range up to 5Hz. This approximate procedure is suggested in FEMA-440 (ATC, 2005) and ASCE41-06 to apply reduction factor to the design spectra.

3.3. Comparison with Instrumented Records

The spatial incoherency model is applied in instrumented buildings with shallow foundations. It was observed that the result gives some approximation in some buildings but the model does not capture the variation in other buildings, particularly where amplifications have been observed.



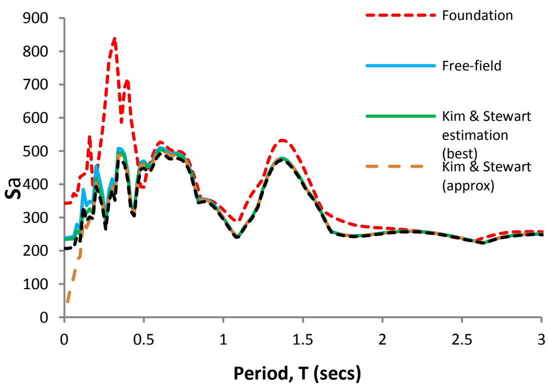


Figure 3.2. Comparisons of response spectra as per ASCE41-06 and Kim and Stewart model with observed records in (a) Los Angeles Office building for Northridge Earthquake in building N-S direction (b) El Centro Imperial Co. Service building for Imperial Valley Earthquake in building E-W direction

Figure 3.2 shows some example cases of modified motion by the procedure compared with observed ones. In Los Angeles 15-story office building, original Kim and Stewart model gives reasonable estimate for foundation motion in the range after 0.5 sec. However, simplified model as per ASCE41-06 and approximate model grossly underestimate the motion up to period of 1.5 sec. In case of El Centro Imperial Co. Service building all the estimates fail to provide reasonable prediction of foundation motion in short period range. As foundation motion is significantly amplified, the model obviously missed to capture the phenomena.

It is reported that Imperial Co. Service building sustained significant damage during the Imperial Valley earthquake despite the level of shaking in nearby free-field is only about 0.15g. The instrumented record shows that the motion gets highly amplified. There are number of other buildings, particularly low-to medium rise with surface foundation which experiences amplifications in one and other earthquakes.

4. WAVE PASSGE EFFECT

Newmark et al. (1977) proposed the numerical averaging procedure to produce modified motion over the foundation slab from free-field. The technique is based on the concept that the averaging of motion happens over a time delay in excitation to parts of foundation caused by the horizontally propagating waves that impinge first on one side of the building foundation and moves to the other side. The averaging is done in effect of transit interval and moved along the acceleration time history.

For a rigid foundation, the kinematic effect in translational motion is quantified by Clough and Penzin (1995) in terms of 'Tau-effect'. If the rigid foundation has dimension equal or more than apparent wave length in frequency range of interest, the motion at the base of foundation will be the average of the free-field motion over the foundation area. For one dimensional horizontal wave propagation, they proposed that the modified translational motion is obtained by Fourier transforming the free-field acceleration and multiplying the amplitude by a complex quantity called 'Tau-factor' and performing inverse Fourier transformation to get the modified acceleration. The transfer function is always less than unity and hence this method cannot capture the amplification of the motions at the base that were observed in some of the instrumented buildings.

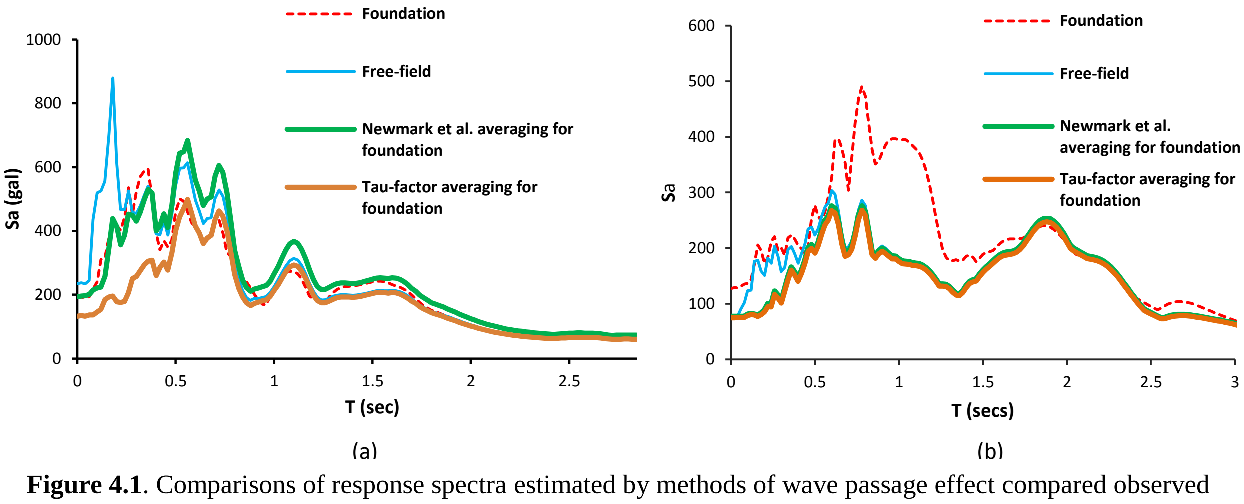


Figure 4.1. Comparisons of response spectra estimated by methods of wave passage effect compared observed record in (a) Hollywood Storage building for Northridge Earthquake in building E-W direction (b) San Bernardino 3-story office building for Landers Earthquake in building E-W direction

The techniques to estimate foundation slab motion describe above were applied were applied in Hollywood Storage Building (Fig. 4.1a) and San Bernardino office building (Fig. 4.1b) for Northridge earthquake and lander earthquake respectively. The averaging technique proposed by Newmark et al. has a good estimate in short period range in Hollywood storage building. However, it overestimates the motion in medium period range. The 'Tau-factor' technique is not in good agreement in short period range as it underestimates the foundation motion. In the case of San Bernardino Office building where foundation motion is amplified in medium period range, none of the above method gives good estimations. This is obvious for 'Tau-factor' technique as their transfer function always give reduction only. In principle, amplification is possible in the numerical averaging technique proposed by Newmark et al. (1977), which however could not capture the motion and follows the same estimate as the other technique provides.

5. CONCLUSION

Out of 99 pairs of records of instrumented buildings with foundation with rigid slab or interconnected strip footing resting on ground (shallow foundations with no basement), two-thirds of the records in buildings with surface foundations showed a reduction in motion at the foundation level compared corresponding free-field. However, in one-third of those motions, we observed amplification of motion at foundation level compared to free-field. ASCE approach of calculating motion reduction at the foundation level does not produce reasonable estimate of motion variation. Other approaches of calculating the variation motion also don't correspond to the observed data. The basis of all methods for quantifying the variation of motion between foundation base and nearby free-field as described above is the averaging effect of wave passage or spatial incoherency which always produce the reduction of motion at the at the foundation level.

The ASCE approach which accounts only reduction of motion at the foundation needs to be closely reviewed from all aspects including the basis of averaging model it used. A clear picture of the mechanics of interaction should be established, which would justify the variation of motion, both with amplification and de-amplification. The case of amplification is more important considering the fact that the analysis using the current procedure could significantly underestimate the demand to the structure. A more detail study of observed cases of amplification of motion at foundation level in one earthquake and reduction in another earthquake and the case where motion gets amplified in one direction and reduced in another direction could lead us to better understanding of the phenomena.

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