Estimating Total System Damping for Soil-Structure Interaction Systems

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For a realistic soil-structure interaction (SSI) analysis, material damping in the soils and structural materials as well as the foundation radiation damping should be considered. Estimating total system damping is often difficult due to complex interplay of material damping and radiation damping in the dynamic solution. In practice, however, an estimate of total system damping is frequently needed for evaluation of SSI effects and for detailed linear or nonlinear structural analysis in order to develop realistic results. The simple methods typically used to estimate structural damping from the dynamic response of the structure often fail to yield realistic system damping mainly due to frequency dependency of the foundation stiffness and dashpot parameters. In this paper a summary of series of parametric studies is discussed and an effective approach to estimate system damping for SSI systems is presented. The accuracy of the method is verified using a model of a large concrete structure on a layered soil site.

INTRODUCTION

Regulations for the seismic design of Nuclear Power Plants permit soil-structure interaction (SSI) analyses in the frequency domain, with the full effects of radiation damping, without any limitations. The frequency domain solutions are generally more suitable for incorporation the damping effects since these solutions incorporate the frequency dependency of the foundation stiffness and damping rigorously and can handle the far field boundary conditions more accurately. There are numerous publications reporting the foundation stiffness and damping for surface or embedded foundation on uniform halfspace or layered sites using the frequency domain approach.

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On the other hand analyses in the time domain, and particularly modal analysis that requires specification of damping for each mode, had limits of 15 % or even 10 % imposed on the damping. Because radiation damping could be significant in some cases, leading at times to effective values of damping in the first mode of 20 % or 25 % under horizontal excitation and up to 50 % or more in vertical vibration, the results of both types of analyses could be very different, with the time domain solution overly conservative. There was little interest or incentive in finding what the effective values of damping implicit in the frequency domain approach were or what should be the values of modal damping to be used in the time domain models to yield similar results. Analyses in time and frequency domain cannot produce identical results because each one involves different approximations. One can obtain, however, very similar and reasonable results if consistent assumptions are made and the values of the different model parameters (frequency independent foundation stiffness, damping, etc.) are wisely selected. To do this it is necessary to look in more detail at the effective damping implicit in frequency domain SSI analyses.

Currently dynamic non-destructive testing is increasingly used to assess the condition of existing structures for health monitoring and damage assessment. The structure may be excited by very small amplitude dynamic loads, by ambient vibrations or by actual earthquakes. Its characteristics are to be determined from the recorded motions at various points where sensors are installed. These characteristics are often expressed in terms of the natural frequencies, mode shapes and modal damping values, which may vary in time depending on the level of excitation. The experimental determination of damping values for multi-degree of freedom systems without a unique, clearly defined, source of energy dissipation represents a problem similar to that encountered when attempting to specify modal damping for SSI analyses in the time domain.

The objectives of this work are to explore the effective values of system damping implicit in SSI systems in the frequency domain, to compare the results of different procedures to estimate damping from response records, and to compare the results of SSI analyses in the frequency domain with those of time domain solutions using realistic parameters. The emphasis is placed on estimating the total system damping for the key dynamic structural responses that inherently include the effects of material damping in the system, the radiation damping due to the SSI effects recognizing complex contribution of the SSI modes and the structural deformation modes in the response. In this paper first the types of damping and modeling of damping for dynamic analysis are discussed. Next the simple methods typically used to estimate damping from structural responses are discussed. A series of parametric studies are performed and the results are discussed to evaluate the merit of each method to estimate total system damping. From the parametric study, the most effective method is identified. The accuracy of the method is tested by applying it to a lumped parameter SSI model to estimate system damping using the time integration method and comparing a key response to the complete SSI solution of the problem. Unless otherwise noted, all computer analyses in this paper are using SASSI2000 (Lysmer et. al, 1999) computer program.

DAMPING AS A MEASURE OF ENEGY DISSIPATION

Treatment of damping as a means to model energy dissipation starts in structural dynamics texts by considering a single degree of freedom system with a viscous dashpot. The dashpot has a constant 'c' and a resisting force directly proportional to the rate of deformation (the relative velocity of the mass with respect to the base). This is often referred to as linear viscous damping. One can define a fraction of critical damping β as

$$\beta = \frac{c}{2\sqrt{km}} = \frac{c}{2m\omega_0} = \frac{c\omega_0}{2k} \tag{1}$$

where k is the stiffness of the system, m the mass and ω_0 the undamped natural frequency of the system. When dealing with this damping the physical constant is the dashpot value 'c'. The value of β is not only a property of the dashpot but also depends on the rest of the system. It can be easily seen that for a fixed c, defining a dashpot, if both k and m vary proportionally, maintaining the natural frequency constant, the fraction of critical damping will decrease with increasing k and m; if m is maintained constant and k is varied, changing the natural frequency, the value of β will decrease with increasing natural frequency (mass proportional damping); if k is kept constant and m varies, changing again the natural frequency, β will increase with frequency (stiffness proportional damping).

It should be noticed that in reality viscous forces (such as drag forces induced by motions in a fluid) are often proportional to the velocity raised to a certain power and are therefore nonlinear. More importantly unless one attaches actual viscous dampers at different points of the structure, most of the energy dissipation in structures does not occur in the form of linear viscous damping. This model is used primarily because it leads to a linear differential equation that can be easily solved analytically. It is, however, commonly accepted and most engineers tend to think of damping in terms of the fraction of critical damping. Alternative forms are frictional (Coulomb) damping, hysteretic damping associated with nonlinear behavior and hysteresis loops in the force displacement relation of the stiffness, and radiation damping due to radiation of waves in a continuous medium away from the area of the excitation. A mathematical idealization, without a clear physical model, is the linear hysteretic damping D (sometimes referred to as structural or material damping). It tries to simulate the behavior of a hysteretic nonlinear system under steady state vibrations with fixed amplitude (the value of the damping would be a function of the amplitude). The linear hysteretic damping is defined as

$$D= E_d/ (4\pi E_s) \qquad (2)$$

where E_d is the energy dissipated per cycle (area of the hysteresis loop) and E_s is the maximum strain energy (assuming an equivalent linear system with the secant stiffness and the same amplitude of vibration). This damping is then included in dynamic analyses (or wave propagation studies) in the frequency domain using complex moduli of the form E(1+2iD) or G(1+2iD) where E and G are the Young's and shear modulus of the material. This is what is commonly done to model the soil in soil amplification or soil structure interaction analyses with most of the available software in the public domain. The damping D is independent of frequency. Considering instead the cyclic behavior of a system with linear viscous damping and the same amplitude of vibration, and applying the above formula one would find that in that case

$$D=\beta\ \omega /\omega_0 \qquad (3)$$

where ω is the frequency of the steady state vibration and ω_0 is the natural frequency of the single degree of freedom viscous system. This implies that to simulate the effect of viscous damping with a linear hysteretic model D would have to increase proportionally with frequency and to simulate hysteretic (frequency independent) damping with a linear viscous system β would have to decrease with increasing frequency. Linear hysteretic damping is only properly defined in the frequency domain and under steady state vibrations although it is used for transient dynamic analyses with the Fourier transform. Since damping is particularly important at or near resonance it is common to make simply $D=\beta$. This would result in two systems with the same amplitude of response at resonance but different behavior at other frequencies.

In SSI problems energy is dissipated in the structure through friction and nonlinear behavior and in the soil through nonlinear behavior and radiation. To arrive at an effective value of damping for the complete system it is necessary to combine these different contributions. The internal damping in the structure is often assumed to be viscous although a hysteretic model would be more realistic. With viscous damping its contribution to the damping of the complete system is multiplied by the ratio of the combined natural frequency to that of the structure by itself on a rigid base raised to the cube. For the hysteretic case the factor would be only squared. The internal soil damping is normally considered using linear hysteretic damping (complex moduli) with analyses in the frequency domain. For time domain analyses it is common to use Rayleigh damping attempting to maintain it nearly constant and close to the desired value over the range of frequencies of interest. When a steady state harmonic load P is applied on top of a rigid mat the resulting displacement will reach after a short while a steady state condition. In this range the displacement will have an amplitude U and will be out of phase with the applied force by an angle φ (or a time lag $\tau = \varphi/\omega$ if ω is the frequency of vibration). It is common to express the foundation stiffness in the form

$$k = k_{real} + i\, k_{imag} = P/U \cos\varphi + i\, P/U \sin\varphi \qquad (4)$$

where the ratio P/U and the angle φ are in general functions of the frequency. By analogy the dynamic stiffness of a single degree of freedom system with linear viscous damping would be

$$k_{dyn} = k - m\,\omega^2 + i\,\omega\, c \qquad (5)$$

and for a system with hysteretic damping

$$k_{dyn} = k - m\,\omega^2 + 2i\, D\, k \qquad (6)$$

It should be noticed that for the system with linear viscous damping the imaginary part of the dynamic stiffness increases proportionally with the frequency of vibration. The plot of imaginary stiffness versus frequency would be a sloping straight line. Dividing it by ω one obtains a horizontal line (independent of frequency) with the value of the dashpot constant c. For the linear hysteretic system on the other hand the imaginary part is constant and dividing it by the frequency one gets a hyperbola with very large values for low frequencies and tending to 0 as the frequency increases. A system with both viscous and hysteretic damping would have an imaginary stiffness consisting of the sum of a constant and a sloping line with slope c. Dividing it by ω would yield a hyperbola tending to a constant value c.

When applying horizontal harmonic forces to a rigid mat foundation on the surface of an elastic half space the real part of the stiffness is essentially constant (it actually has a small variation with frequency) and the imaginary part is essentially a straight line. This implies that the foundation can be modeled as a spring and a viscous dashpot. If the soil had some internal damping, of a hysteretic nature, the imaginary part of the stiffness would be again the sum of a constant and a term linearly proportional to ω and dividing it by ω would result in a hyperbola. The limiting value of the hyperbola as the frequency increases represents the radiation damping. When applying on the other hand a vertical force to the foundation if the soil has a Poisson's ratio of the order of 0.4 or higher the real part of the stiffness looks like a second degree parabola with negative curvature suggesting a model with a spring and a mass (added mass of soil). In this case the dynamic stiffness can become negative for high frequencies much as the value of $k\text{-}m\omega^2$ would become negative for a single degree of freedom system. In attempting to define the effective damping for a rigid block placed on top of the foundation one should add the mass of soil to that of the block and consider the static stiffness instead of using a zero or negative stiffness. For a foundation on the surface of a soil layer of finite depth (resting on much stiffer, nearly rigid rock) the real and imaginary parts of the stiffness will exhibit fluctuations associated with the natural frequencies of the layer. For a soil without any internal damping the stiffness would become 0 at the soil natural frequency. Below a threshold frequency (the fundamental frequency of the soil layer in shear for the horizontal case, the corresponding frequency in compression-dilatation for the vertical and rocking cases if Poisson's ratio is 0.3 or less, and an intermediate frequency for higher Poisson's ratios) there will be no radiation and the damping will be associated only with the internal, hysteretic, dissipation of energy in the soil. Above the threshold frequency there will be radiation and the results will be similar to those of the half space except for their fluctuations. The interpretation of what is the effective damping is more difficult for these cases.

MEASUREMENT OF DAMPING

Measurement of damping is carried out either through free vibration or forced vibration steady state tests. Under free vibrations a system with linear viscous damping experiences an exponential decay in amplitude. The natural logarithm of the ratio of the amplitude of a peak to that of the next one of the same sign would be then $2\pi\beta/(1\text{-}\beta^2)^{0.5}$ or approximately $2\pi\beta$ for low values of damping. The logarithm of the ratio of the amplitude of one positive peak to that of the next negative one (or valley) would be half. The logarithm of the ratio of the amplitude of a peak to that of the peak n cycles later would be n times this quantity. If the damping is not of a linear viscous nature the ratio of the amplitudes of two consecutive peaks would not be constant. In laboratory free decay tests it is common to observe a variation in this ratio and to take an average over various cycles. Because these free vibrations take place at the natural frequency of the sample one could assume that the measured β can also be D.

In laboratory steady state cyclic tests at a given frequency one can obtain the force deformation diagrams for each cycle and compute the energy dissipated (area of the hysteresis loop) and the equivalent secant stiffness (to compute the maximum strain energy). The damping ratio D can then be directly calculated. This is what is normally done to determine for different soils the variation of the effective shear modulus and damping with the level of shear strains (and frequency in some cases). An alternative is to determine experimentally the response of the sample to harmonic excitation with different frequencies, plotting the displacement amplitude (divided by the amplitude of the applied force) versus frequency. This is the traditional amplification function for the response of a single degree of freedom system to a harmonic steady state excitation. The peak in the response occurs at a frequency $\omega_0 \, (1-2\beta^2)^{0.5}$ or approximately ω_0 (undamped natural frequency) for low values of damping. Its value is $1/2\beta(1-\beta^2)^{0.5}$. The value of the amplification at the frequency ω_0 would be exactly $1/2\beta$. It is common as a result to measure the amplitude of the peak U and to calculate the damping as 1/2U. Because the exact peak may be difficult to obtain an alternative is to use the half power bandwidth method (Clough and Penzien, 1993, Chopra, 1995). In this case calling ω_2 and ω_1 the frequencies at which the amplitude would be $\frac{1}{\sqrt{2}}$ U the damping can be obtained approximately (again for low values of damping) as $\beta = (\omega_2 - \omega_1)/(\omega_2 + \omega_1)$. These expressions assume again linear viscous damping and a single degree of freedom system. When dealing with experimental frequency response curves obtained in the field (either applying very small amplitude harmonic excitations, from records of ambient vibrations, or from records of response to actual earthquakes) it is common to use this approach to determine the effective damping in each mode. It is common to assume that the first peak is only affected by the first mode, the second by the first 2 modes, and so on. The fact that it is no longer a single degree of freedom system and that the damping is not primarily of a viscous nature make the reliability of the estimates somewhat questionable.

In SSI problems if a rigid mass M resting on a mat foundation on the surface of an elastic halfspace is subjected to horizontal excitation, calling k_{real} and k_{imag} the real and imaginary part of the foundation stiffness and $c=k_{imag}/\omega$ based on the previous considerations, the fraction of critical damping for the system

$$\beta = \frac{c}{2\sqrt{kM}} \tag{7}$$

would be approximately constant if M is constant. On the other hand if M changed so as to change the natural frequency of the system with $k=M\omega_0^2$, β would increase linearly with the natural frequency and become

$$\beta = c\omega_0 /2k \tag{8}$$

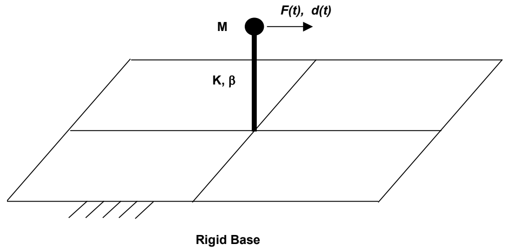
If the soil had some internal damping of a hysteretic nature β so defined would look like a hyperbola as a function of frequency with very large values at low frequencies. It would be more logical then to separate first the hysteretic component (corresponding to the value of k_{imag} at low frequencies divided by 2 k, then apply the above equation to the remaining c and add both results. When dealing with vertical vibration and a soil with Poisson's ration of 0.4 or more one should use the static value of the real stiffness and add to the rigid mass M the added mass of soil in order to estimate the damping (rather than dividing by a k that could become 0 or negative).

When dealing with a soil layer of finite depth the interpretation of the damping becomes more difficult because of the fluctuations in the real and imaginary parts of the stiffness with frequency. One could use the value of the variable k at each frequency or consider instead the static value and consider the difference between the static and the dynamic values an added mass of soil multiplied by the square of the frequency, adding it to the value of the rigid mass.

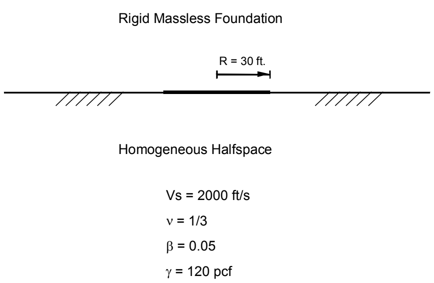
It is noted that other simple relationships have been developed to estimate system damping for SSI systems on the frequency by frequency basis involving structures with single degree of freedom such as those developed by Roesset (NUREG/CR 1780, 1980). However, the purpose of this paper is to develop a simple method to estimate the total system damping as it relates to the final dynamic structural responses (such as the acceleration response spectra at selected mass points). Such responses obviously include the effects of material damping in the soil and structure, radiation damping of the foundation and the combined modal effects of structural deformation as well as rigid body SSI motion. As shown in this paper, a more reliable way to estimate the system damping would be to subject the system to an impact load and study its free vibration and assess the damping from decay of the free vibration response.

PARAMETRIC STUDY

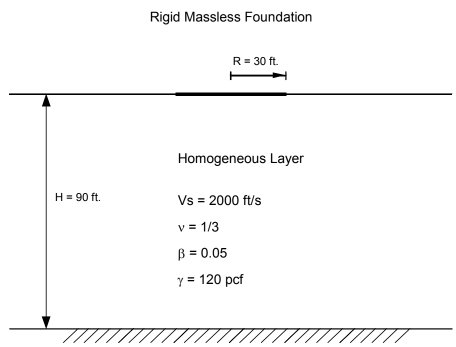
A total of five simple systems shown in Figure 1 have been analyzed. The systems considered are as follows. A single degree-of-freedom (SDOF) system consisting of a lumped mass and a spring as depicted by Case 1 in Figure 1 was analyzed. The base of the model is fixed and has a fixed base natural frequency of 4 Hz. The material damping used is of hysteretic type. Damping values of 5, 10, 15, and 20% are considered. For each material damping value, a fixed base SASSI analysis was performed and the transfer function of the response was obtained. The SDOF of system was also subjected to an impulse load. The impulse load has a unit amplitude and duration of 0.01 second, as shown in Figure 2. The transfer functions of the SDOF system from harmonic seismic analyses and the impulse response functions due to impulse load are shown in Figures 3 and 4, respectively. The transfer function is the amplitude of the total acceleration response of the mass point subjected to the harmonic input acceleration with amplitude of unity. As expected, the peak of transfer function takes place at the natural frequency of the system and its amplitude is a function of the material damping used in the model. The impulse response function is the displacement time history of the response of the mass point subjected to the impulse load. The rate of decay in the displacement response is a function of the material damping used in the model. The half-bandwidth method and the peak of the transfer functions were used to back-calculate the system damping. The impulse response functions from the impulse load were used in conjunction with the decay method to estimate the damping. The summary of the results is shown in Table1. As expected, for a SDOF with constant material damping, all methods predict accurate results close to the material damping used for the model.



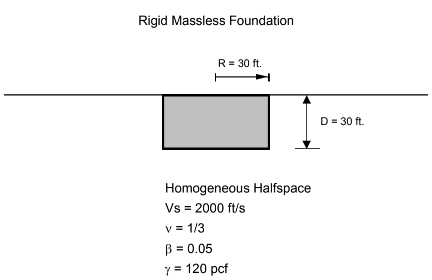
Case 1



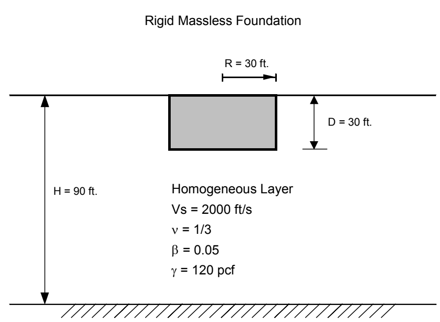
Case 2



Case 3



Case 4



Case 5

Figure 1. Numerical Models Considered for Parametric Study

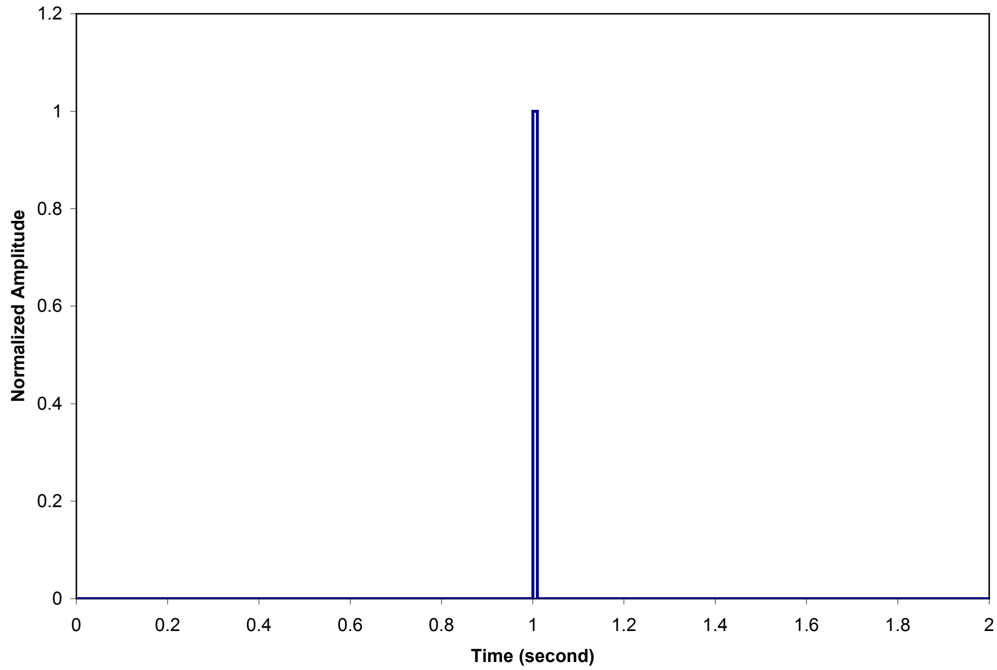


Figure 2. Impulse load to compute impulse response function

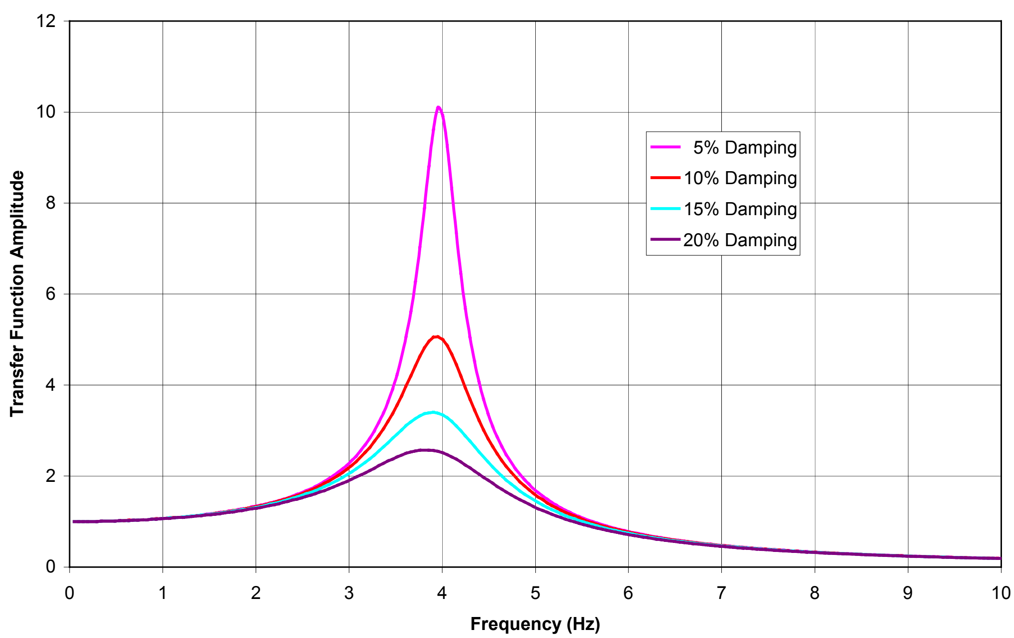


Figure 3. Transfer function results for fixed base SDOF system

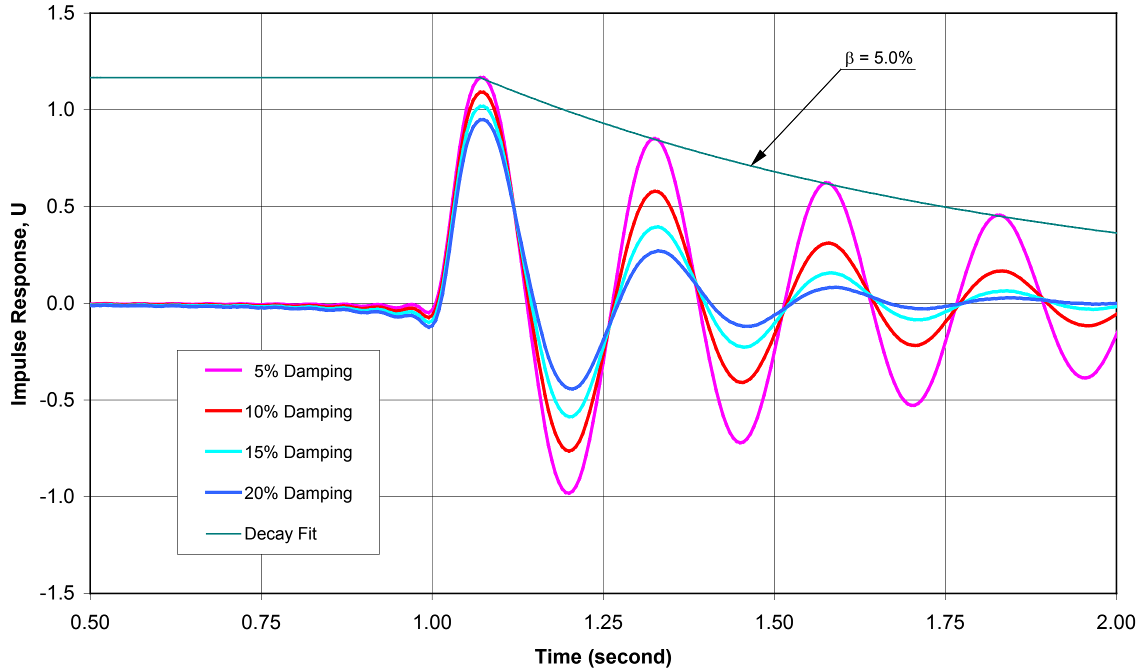


Figure 4. Impulse response for SDOF fixed base system

Table 1. Damping computed for fixed base SDOF system using 3 methods

Approach	Given Structural Damping(%) 5.0	10.0	15.0	20.0
Half Band	4.8%	9.7%	15.9%	22.3%
1/(2*Umax)	5.0%	9.9%	14.7%	19.5%
Decay of Motion	5.0%	10.0%	14.9%	19.6%

Next a surface rigid circular foundation was analyzed. In Case 2 (see Figure 1), the foundation is located on the surface of a uniform elastic halfspace with hysteretic material damping of 5%. In Case 3, the same foundation is placed on the surface of a layered soil resting on rigid base. The thickness of the soil layer is three times the radius of the foundation. Equations 4, 5 and 6 with m = 0 are used to obtain the stiffness (k_{real}) and dashpot coefficients, c. The material damping for soils is included in the SASSI soil properties using the complex soil moduli. In SASSI analysis, a unit amplitude horizontal harmonic load is applied to the foundation and from the real and imaginary parts of the displacement results k and c are computed for each frequency. The results are shown in Figures 5 and 6. For Case 3 the stiffness and dashpot coefficients show a much larger frequency dependency than in Case 2, the uniform halfspace case. Following the impedance analysis, each foundation model was modified by adding a single mass point at the center. A total of 5 mass values were used in separate analyses. The mass values were chosen to have the foundation undamped natural frequencies of 2, 4, 6, 8, and 10 Hz to cover a wide range of natural frequencies. Each foundation system with the mass point described above was analyzed under harmonic seismic loading and was also subjected to the impulse loading shown in Figure 2. The results of analyses in terms of absolute acceleration transfer functions for Cases 2 and 3 are shown in Figures 7 and 8, respectively.

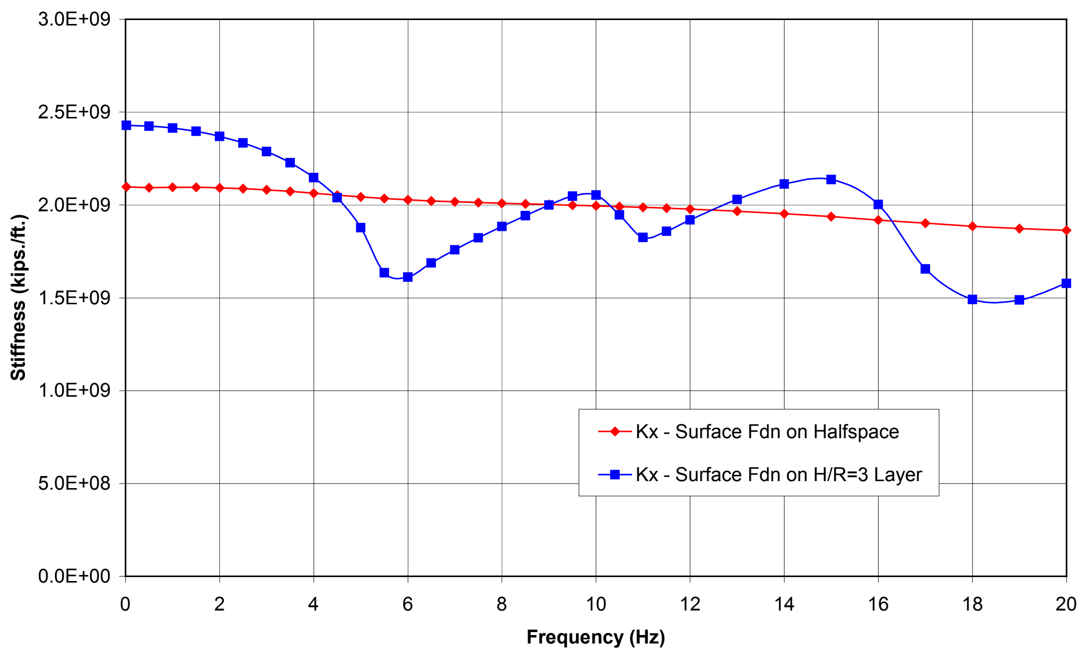


Figure 5. Horizontal foundation stiffness for Cases 2 and 3

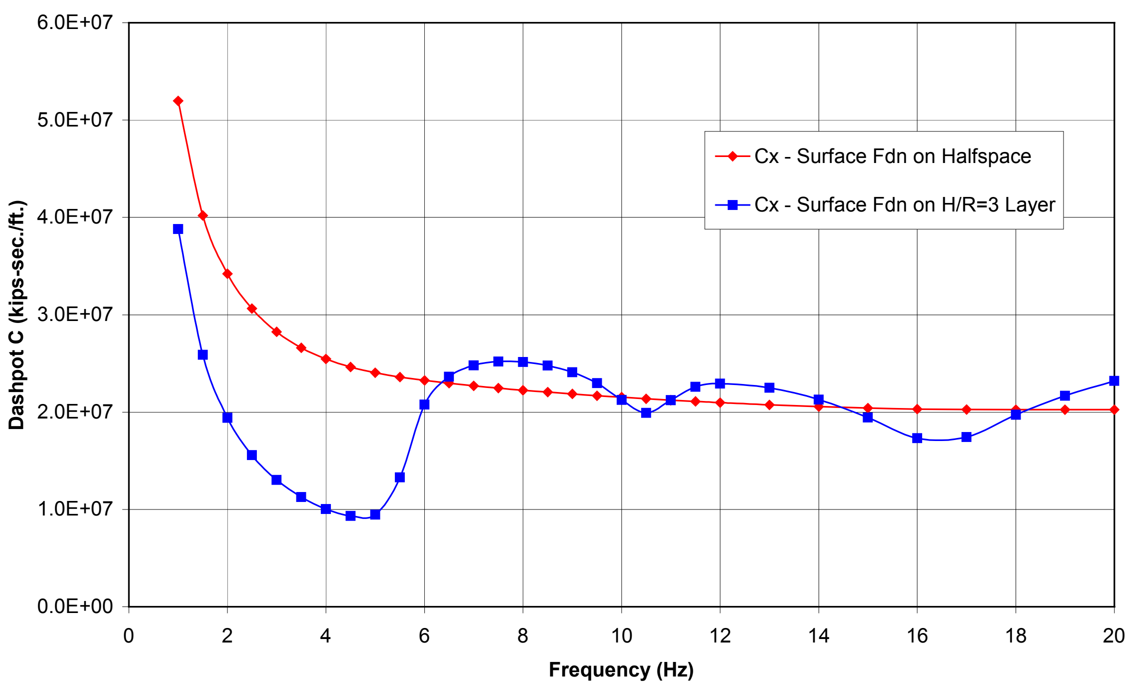


Figure 6. Horizontal foundation dashpot coefficient for Cases 2 and 3

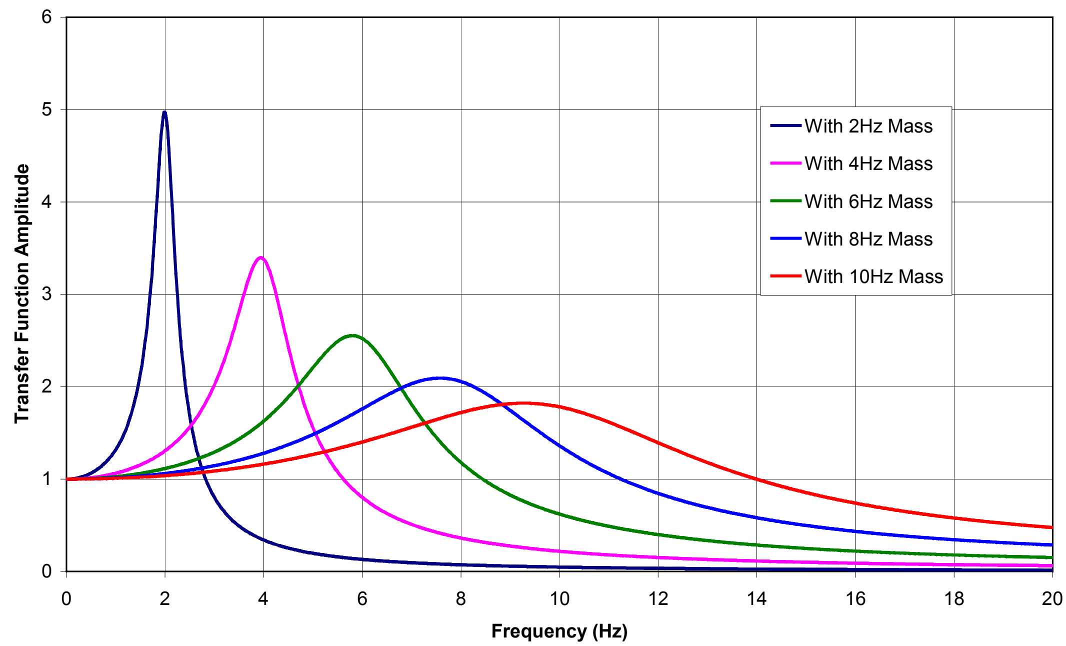


Figure 7. Transfer function amplitude for Case 2 (surface foundation on halfpace)

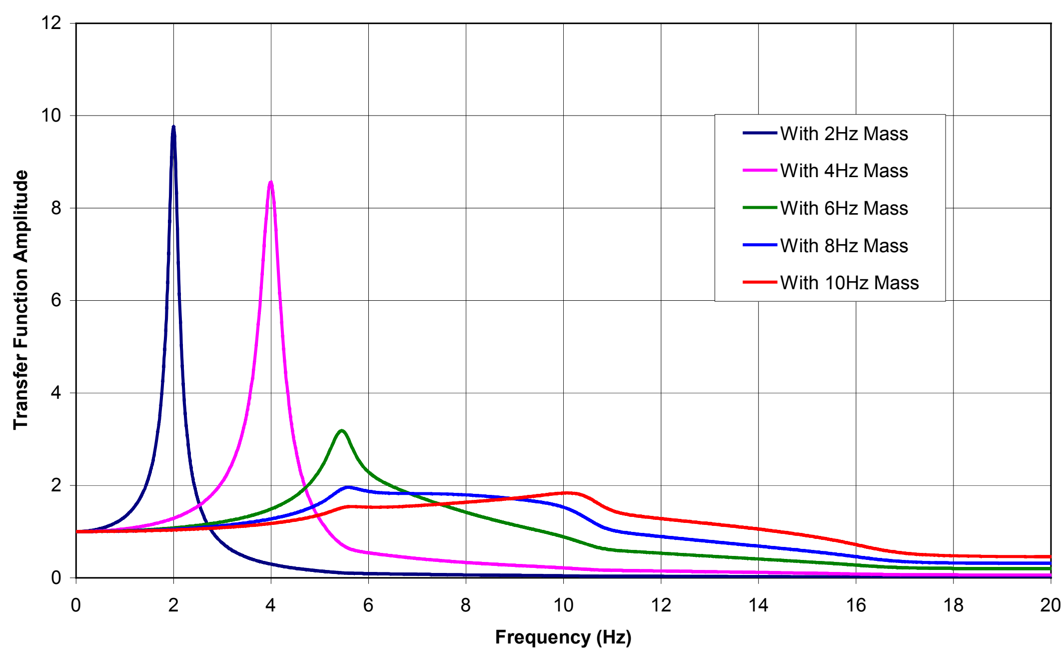


Figure 8. Transfer function amplitude for Case 3 (surface foundation on H/R=3)

The results show the effect of changing the mass points from one value to another. The results in terms of impulse response function are shown in Figures 9 and 10, respectively.

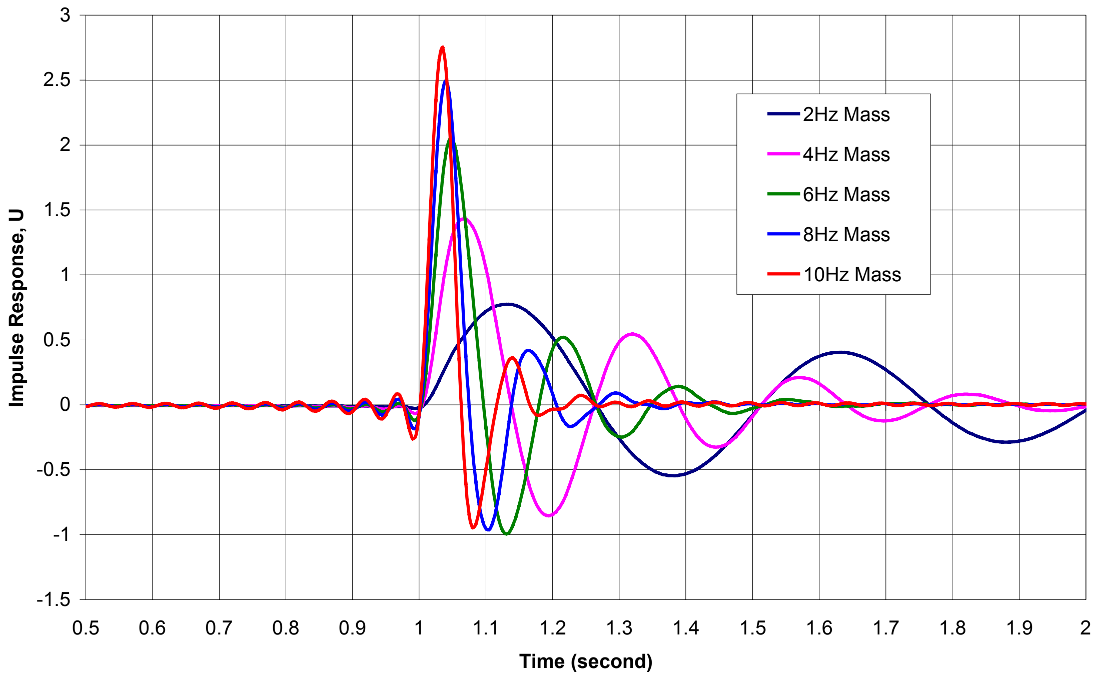


Figure 9. Impulse response functions for Case 2 (surface foundation on halfpace)

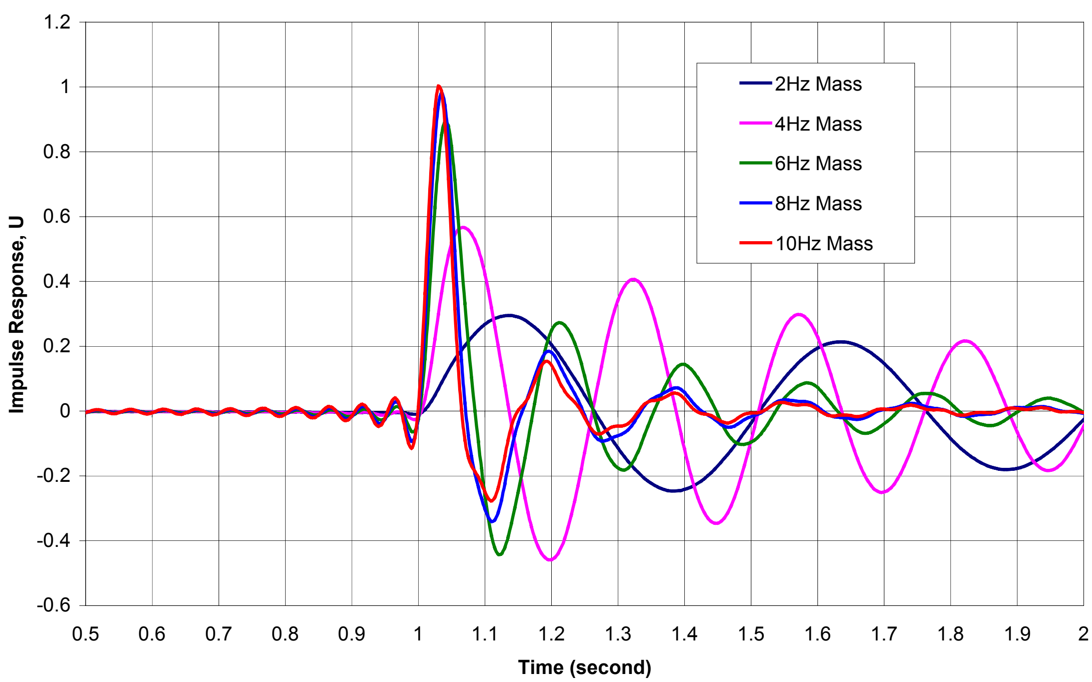


Figure 10. Impulse response functions for Case 3 (surface foundation on H/R=3)

The impulse load analyses were performed for the same 5 values of mass points corresponding to the natural frequencies of 2, 4, 6, 8, and 10 Hz. The estimate of the system damping for the SSI system for both Cases 2 and 3 are shown in Tables 2 and 3, respectively. As shown, the half-bandwidth method loses accuracy for higher natural frequencies and for Case 3 where foundation stiffness and damping show more frequency dependency than Case 2. This is to be expected since the half-bandwidth method is formulated for constant (frequency-independent) stiffness and damping conditions. It also fails to work for the layered system where the transfer function is wide and the peak amplification is small (see Case 3 results for 8, and 10 Hz cases). The method using the inverse of the peak also becomes less accurate for higher damping conditions. However, using the damping ratio equation (Equation 1), the damping results tend to be closer to the decay method. It should be noted that the damping ratio method requires the knowledge of the dashpot value at the natural frequency of the system. This information is readily available for a SDOF system where only one natural frequency exists. Estimating dashpot coefficients for a response that involves multi modes with the dashpot highly dependent on frequency of the vibration becomes much more difficult which reduces the accuracy of this method for real application. This point is illustrated in the results of the case study below.

Table 2. Damping computed for Case 2 (surface foundation on halfpace)

Approach	Assigned Mass Frequency (Hz) 2.0	4.0	6.0	8.0	10.0
Half Band	10.5%	16.0%	23.4%	32.0%	41.4%
1/(2*Umax)	10.1%	14.7%	19.6%	23.9%	27.4%
$C/[2*(km)^{0.5}]$	10.3%	15.5%	21.6%	27.8%	33.9%
Decay of Motion	10.3%	15.2%	21.3%	27.4%	30.7%

Table 3. Damping computed for Case 3 (surface foundation on H/R=3)

Approach	Assigned Mass Frequency (Hz) 2.0	4.0	6.0	8.0	10.0
Half Band	5.0%	5.1%	10.0%	-	-
1/(2*Umax)	5.1%	5.8%	15.7%	25.5%	27.2%
$C/[2(km)^{0.5}]$	5.1%	5.9%	24.3%	33.5%	32.5%
Decay of Motion	5.1%	5.4%	18.1%	28.8%	39.6%

Following the analysis of surface foundation, Cases 4 and 5 shown in Figure 1 for an embedded foundation were analyzed. Similarly, Case 4 is a uniform halfspace case and the Case 5 is a layered site case. The foundation stiffness and dashpot coefficients computed for a point at the bottom center of the foundation are compared in Figures 11 and 12. As expected the stiffness and dashpot parameters show a very smooth variation with frequency for the halfspace case (Case 4). Similar to the surface foundation, the models were modified and single mass points were added at bottom center of each foundation with 5 mass values to replicate undamped natural frequencies from 2 to 10 Hz. Both models were analyzed for harmonic seismic input motion as well as the impulse load. The transfer function results are shown in Figures13 and 14, respectively. As shown in these figures, the peak of the transfer function for foundations with natural frequencies above 7 Hz is too small mainly due to high foundation radiation damping. The amplitudes of the response for the layered-soil case (Case 5) are generally higher than those of the uniform halfspace due to less radiation damping for the layered site (see Figure 12). The results in term of impulse response functions are shown in Figures 15 and 16. A summary of system damping values computed from the dynamic results is shown in Tables 4 and 5, respectively. As shown in these tables, the estimated damping varies significantly from one method to other. Variation is particularly more pronounced for the case of layered soil system. The system damping estimated from the decay method appears to be more realistic even for the systems with high undamped natural frequencies. The accuracy of this method is verified for a real size structure on a layered soil site in the next section.

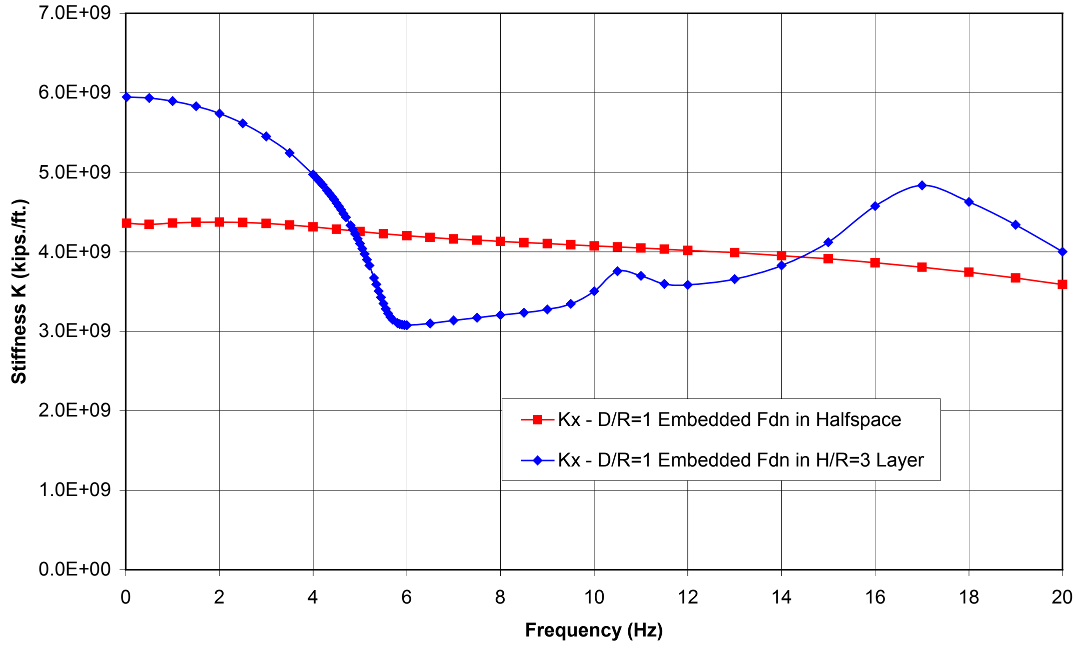


Figure 11. Horizontal foundation stiffness for Cases 4 and 5

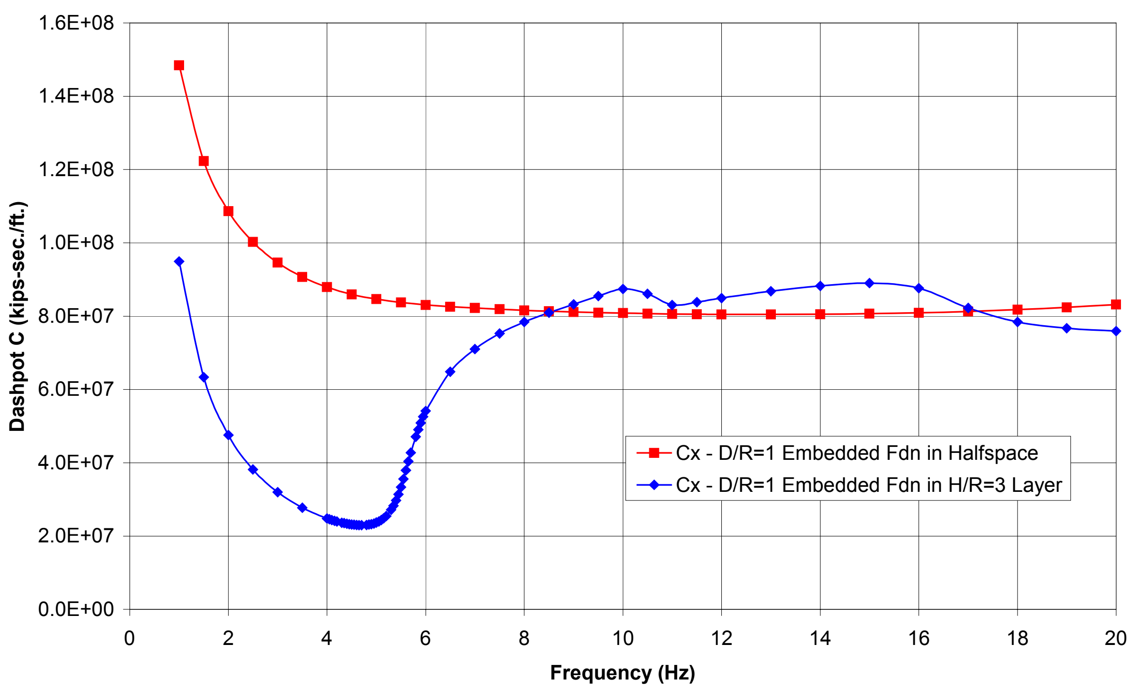


Figure 12. Horizontal foundation dashpot coefficient for Cases 4 and 5

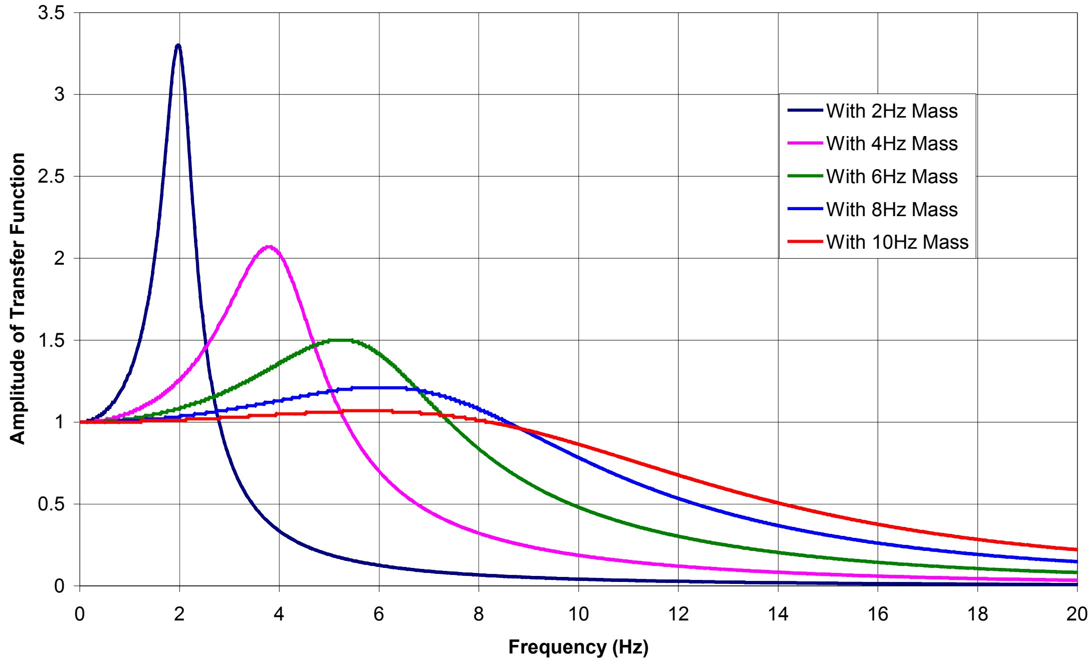


Figure 13. Transfer function amplitude for Case 4 (embedded foundation in halfpace)

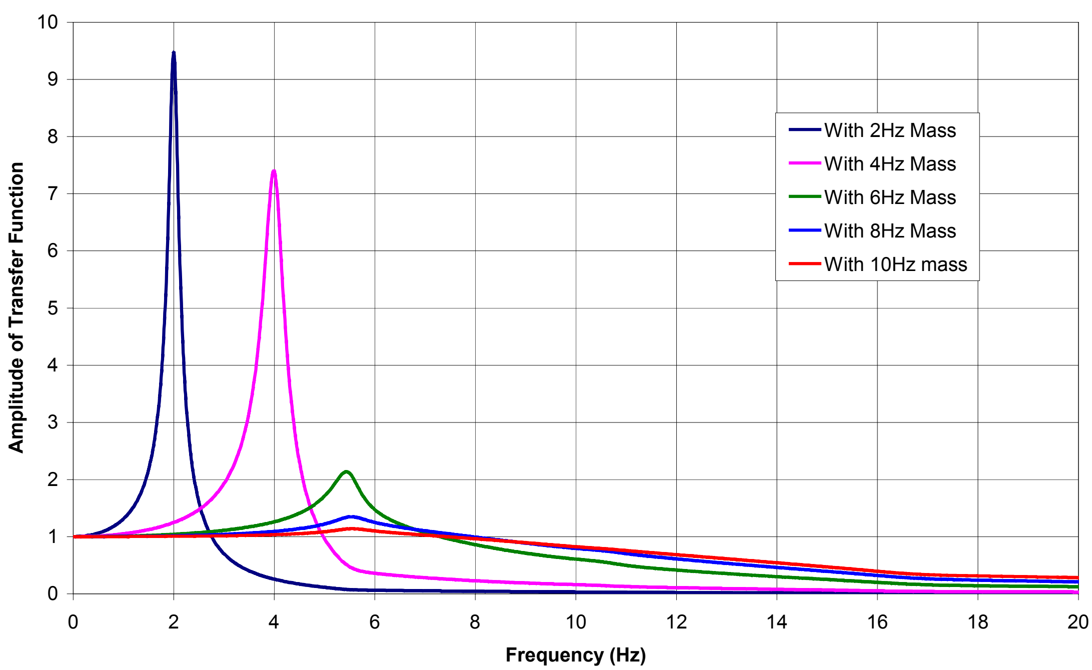


Figure 14. Transfer function amplitude for Case 5 (embedded foundation in layered soil)

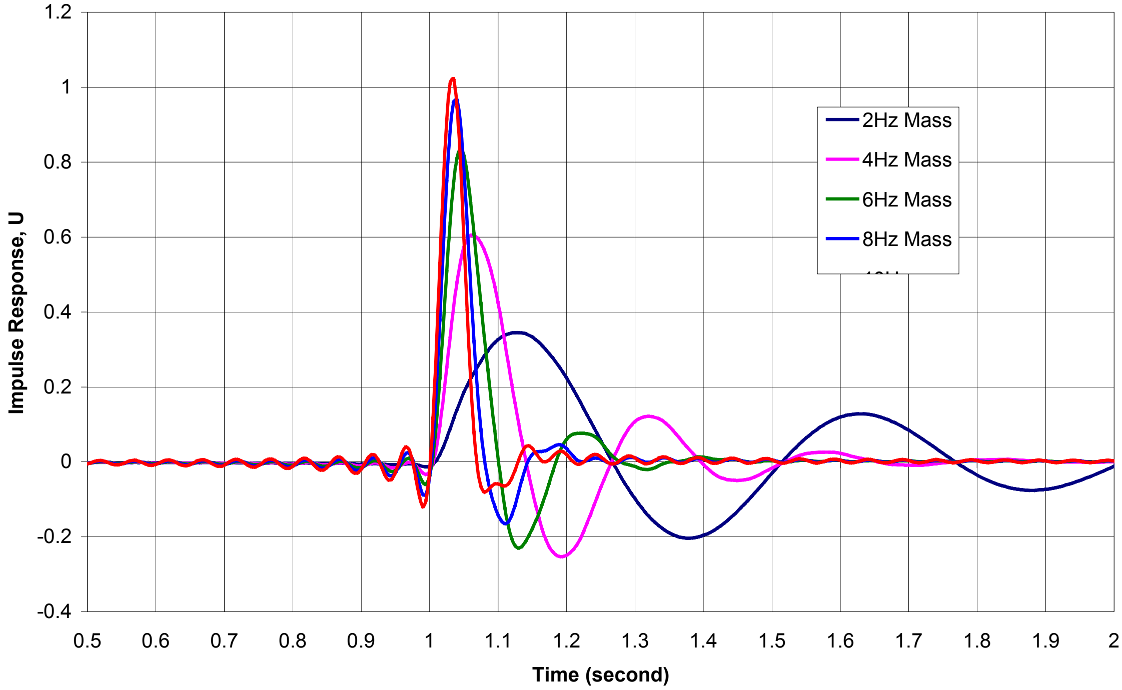


Figure 15. Impulse response functions for Case 4 (embedded foundation in halfpace)

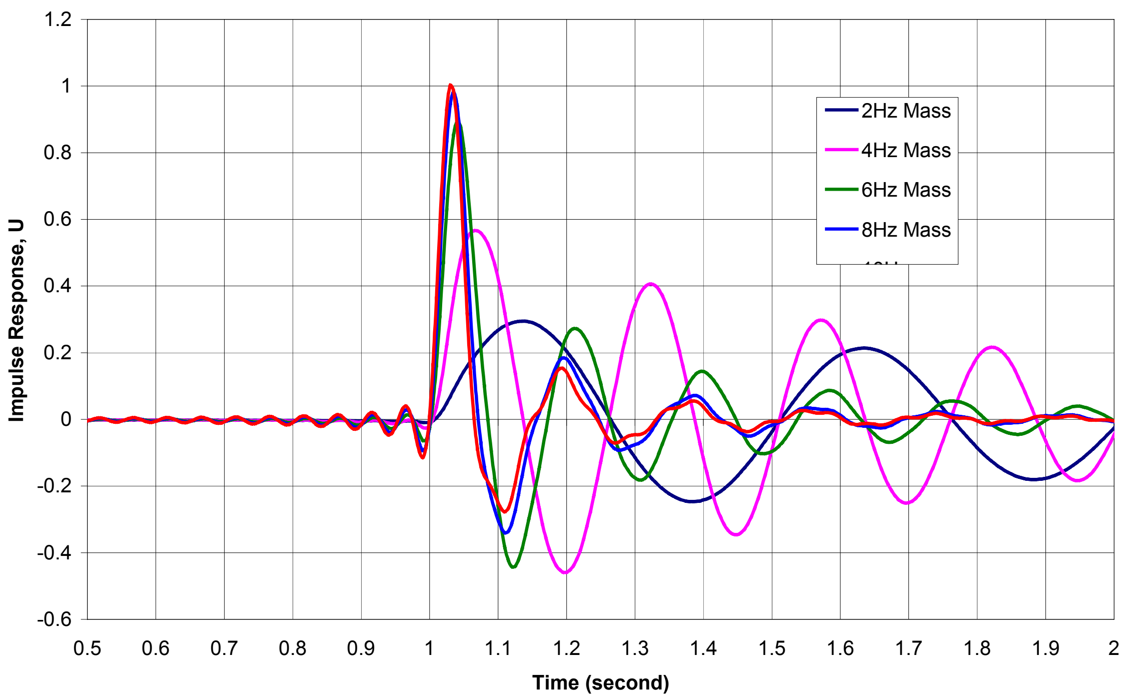


Figure 16. Impulse response functions for Case 5 (embedded foundation in layered soil)

Table 4. Damping computed for Case 4 (embedded foundation in halfpace)

Approach	Foundation Frequency (Hz) 2.0	4.0	6.0	8.0	10.0
Half Band	16.5%	29.8%	61.8%	-	-
1/(2*Umax)	15.2%	24.2%	33.3%	41.3%	46.7%
$C/[2*(km)^{0.5}]$	15.6%	25.6%	37.3%	49.7%	62.3%
Decay of Motion	15.6%	24.8%	35.6%	43.6%	45.2%

Table 5. Damping computed for Case 5 (embedded foundation in layered soil)

Approach	foundation Frequency (Hz) 2.0	4.0	6.0	8.0	10.0
Half Band	5.0%	5.1%	11.7%	-	-
1/(2*Umax)	5.3%	6.8%	23.4%	37.1%	43.9%
$C/[2*(km)^{0.5}]$	5.2%	6.3%	33.2%	61.5%	78.4%
Decay of Motion	5.1%	4.9%	18.6%	25.7%	28.7%

CASE STUDY

In order to evaluate the effectiveness of the system damping obtained from the impulse load method, a dynamic model of a vitrification structure was analyzed. The structure is a concrete shear wall building with a foundation dimension of 322 ft by 253 ft. The major floors in the building are located at Elev. –21 (basetmat), .0, 13, 36, 57 and 86 ft. The SASSI model of the building is shown in Figure 17. Part of the building from ground surface (Elev. .0 ft) to the bottom of the foundation (Elev. –21 ft) was modeled by finite elements and the superstructure was modeled by a beam stick model. This is modeled to include the embedment effect on SSI responses. However to simplify the analysis for this paper, the ground surface was lowered to Elev. –21 ft thus eliminating the foundation embedment. The fundamental fixed base structural modal frequency of the building in the East-West direction is 12 Hz with 70% of the total mass. The stick model is a 3D model and includes eccentricity of the shear and mass centers. A detailed view of the 3D model is shown in Figure 18.

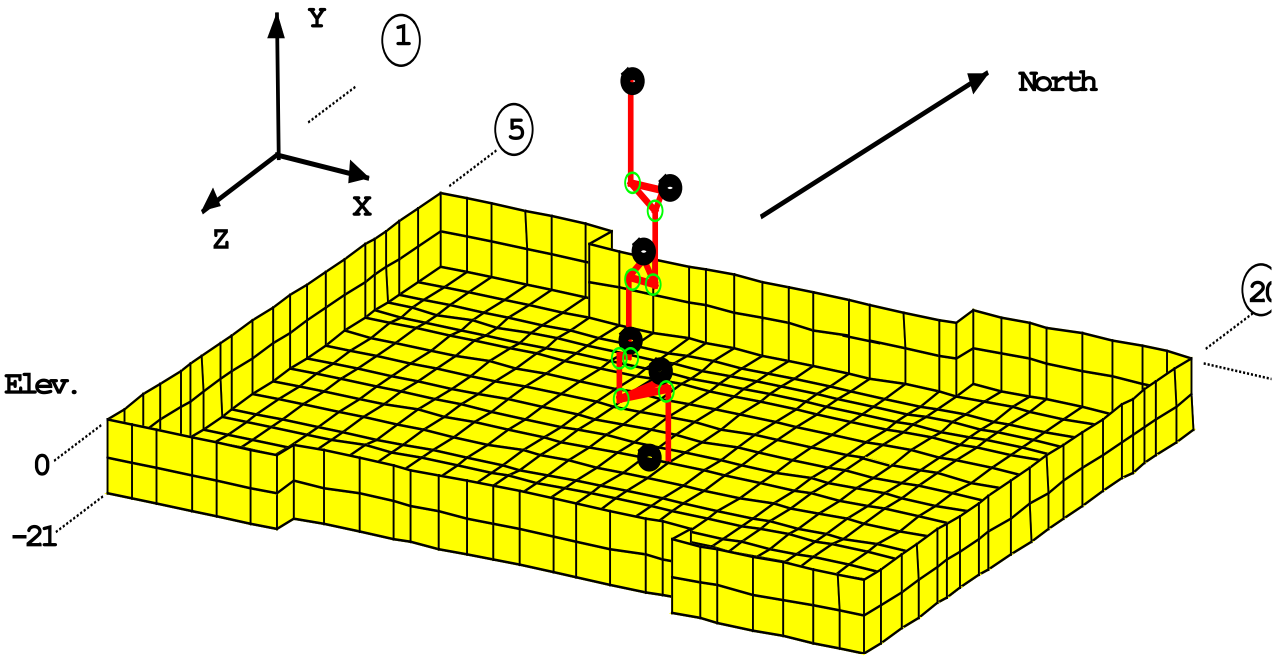
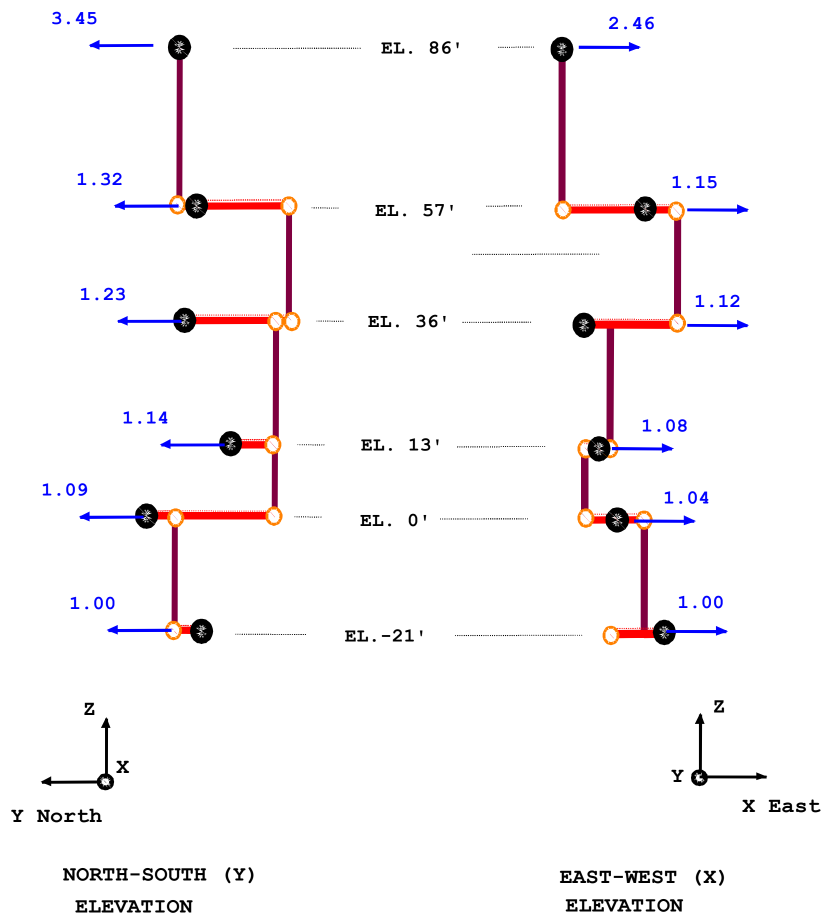


Figure 17. SASSI Hybrid model of the vitrification building



Dynamic 3D Stick Model

Notes: 1. Foundation Mat at El. -21.

2. Impulse loading time histories are applied on lumped mass points. Numbers shown next to the arrows are relative magnitudes proportional to max. acceleration values.

Figure 18. Stick model of the vitrification building

The site consists of very dense layers of sand and gravel with a total thickness of about 300 ft underlain by rigid rock. The strain-compatible shear wave velocity and damping values obtained from free-field SHAKE (Schnabel et al, 1972) analysis are shown in Figure 19. The soils material damping ranges from 2% to 4% depending on the depth of the soil layer.

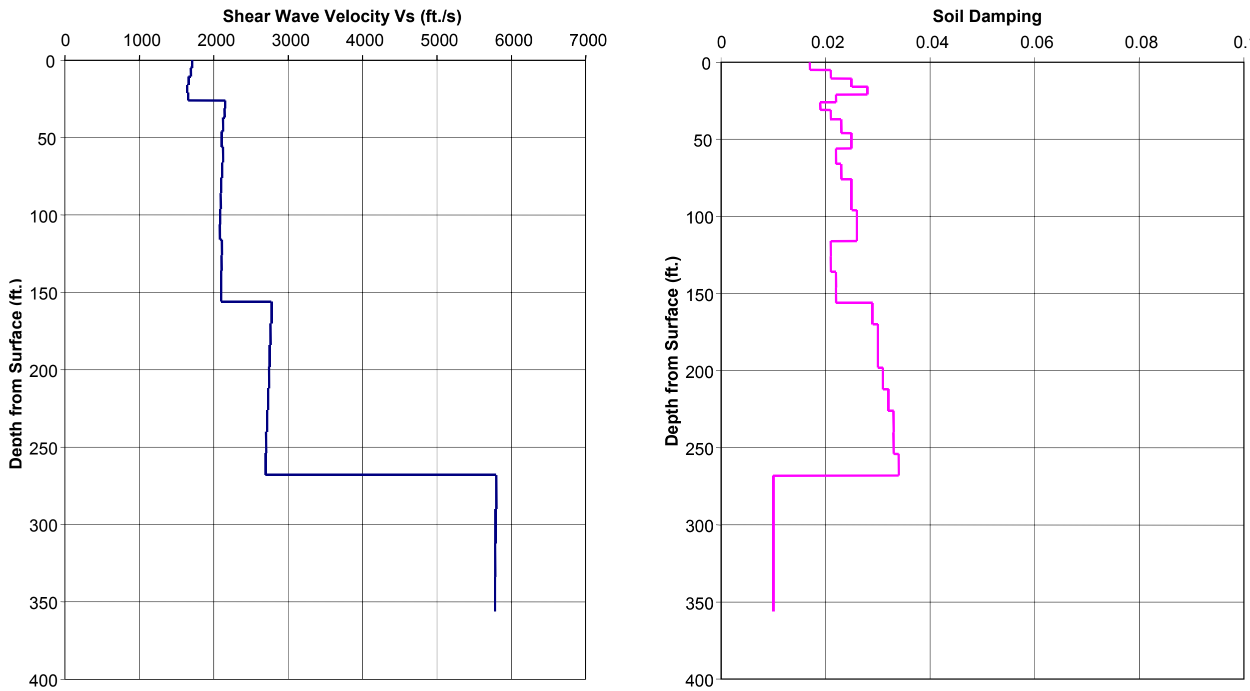


Figure 19. Strain-compatible shear wave velocity and damping profiles

To estimate the system damping first SASSI impedance analysis for the massless rigid foundation was performed and the stiffness and dashpot coefficients were obtained. The results for horizontal translation in the East-West (X-direction in Figure 17) and rocking motion along the North-South axis are shown in Figures 20 and 21. Since the length of the foundation (322 ft in the East-West direction) is about the same as the soil layer thickness (see Figure 19) both stiffness and dashpot parameters show significant variation with frequency.

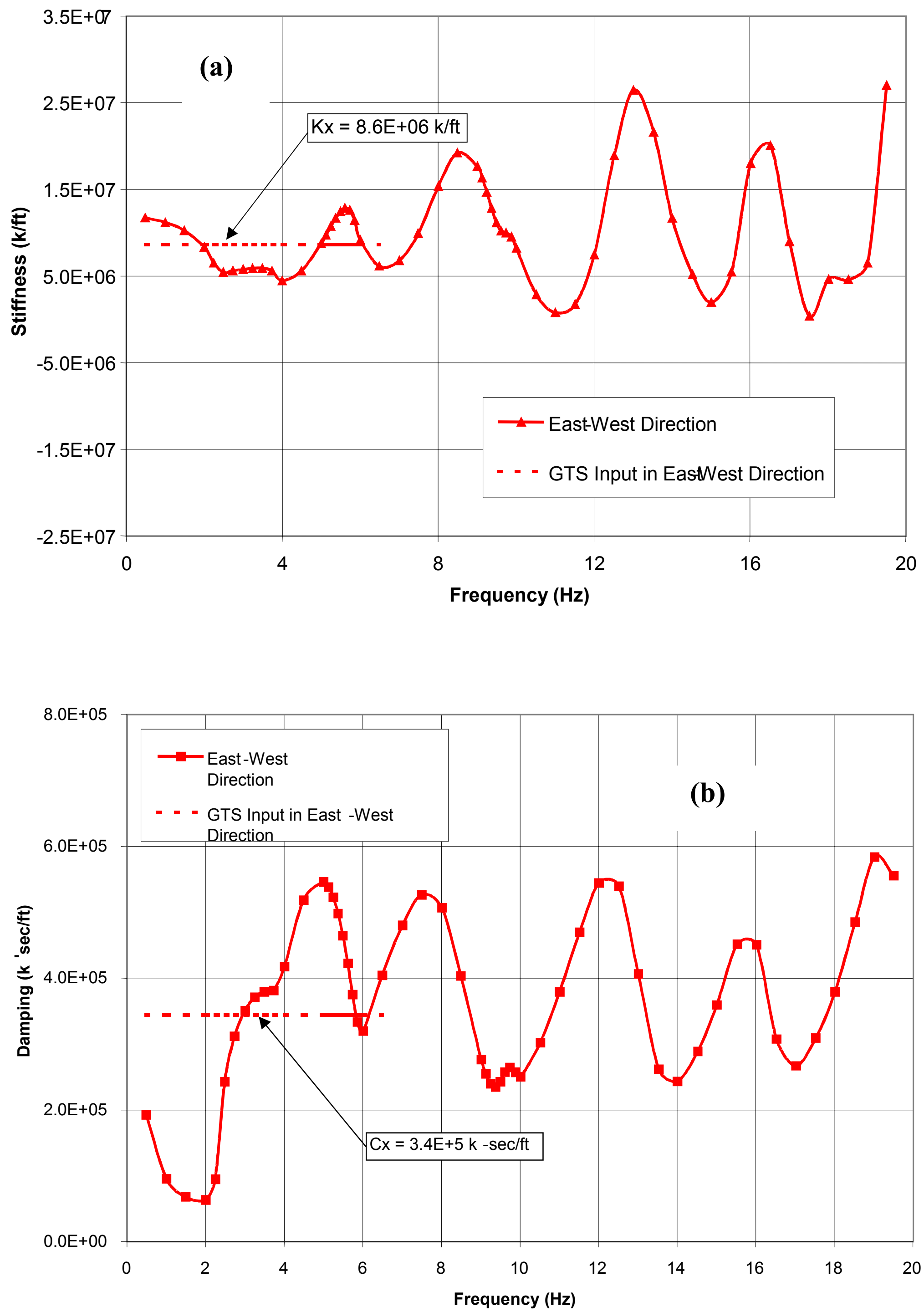


Figure 20. Horizontal foundation (a) stiffness and (b) dashpot coefficients (case study)

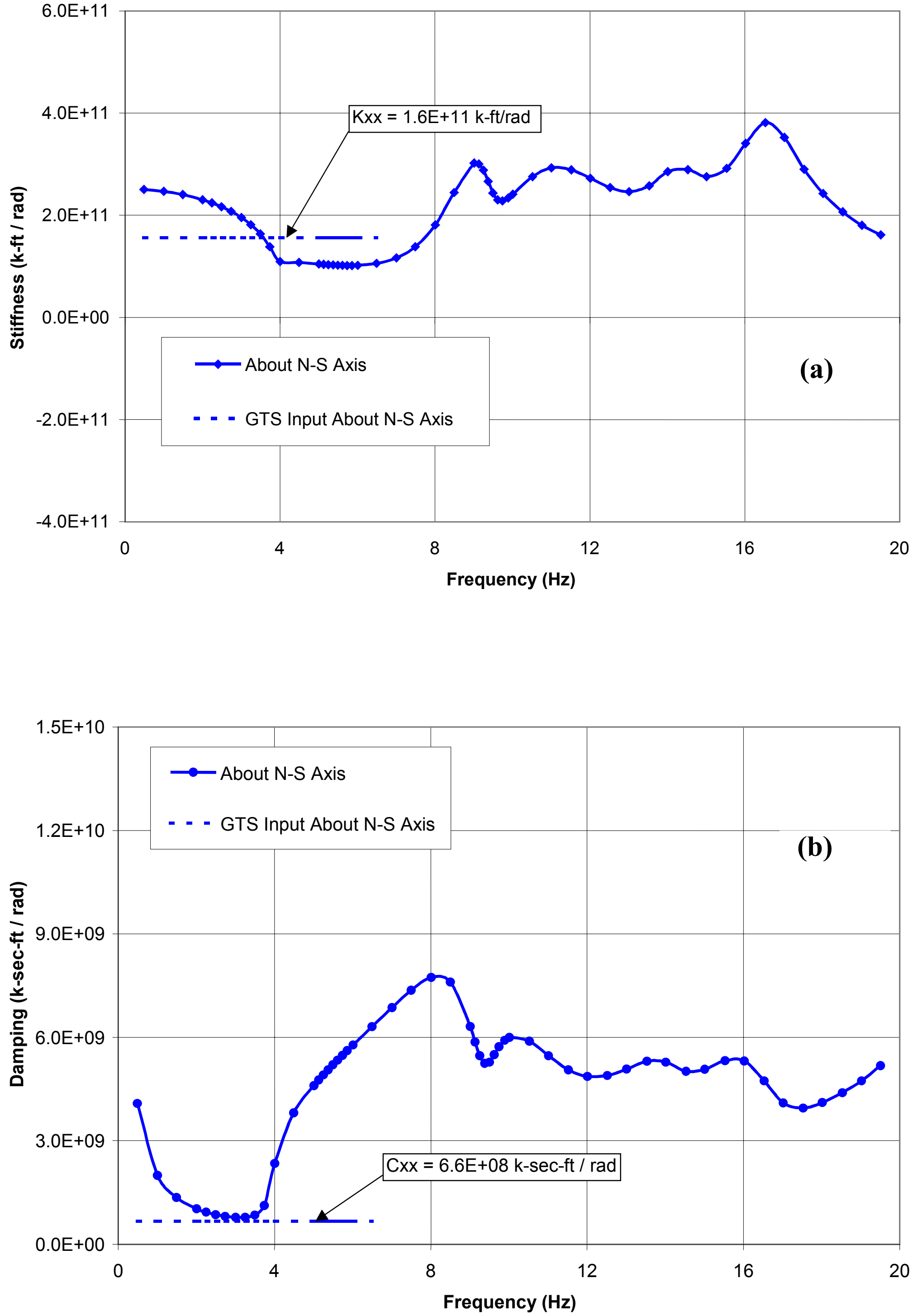


Figure 21. Rocking foundation (a) stiffness and (b) dashpot coefficients (case study)

Following the impedance analysis, SSI analysis of the building was performed using SASSI. The result in terms of amplitude of the total acceleration transfer function is shown in Figure 22. As shown in this figure, the response is controlled by several modes of vibration as evident by the numerous peaks in the transfer function plot. This is a typical response of a multi-story structure on a layered soil system. The peak values are each associated with the foundation stiffness and dashpot that also change with frequency. As shown in Figure 22, it is very difficult to select a particular peak response to use as a basis for obtaining the total system damping. A wrong choice for the peak response amounts to an erroneous system damping.

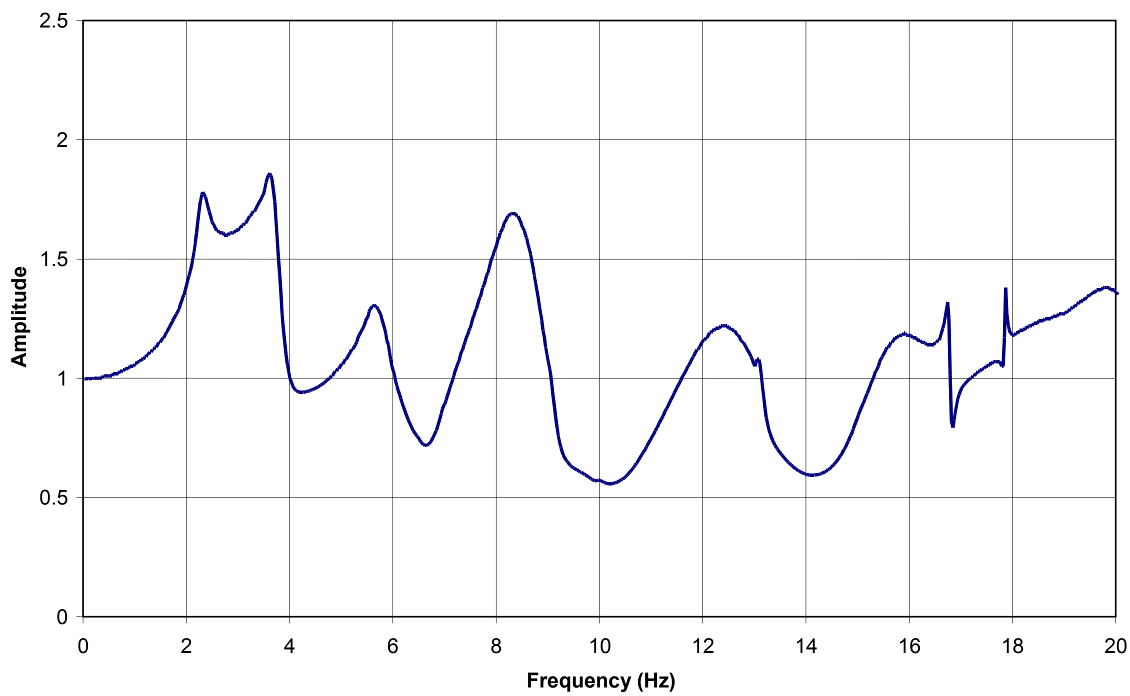


Figure 22. Transfer function amplitude of the node at Elevation 58 ft (SASSI)

To estimate the system damping, the SASSI model (see Figure 17) was subjected to impulse load at all mass points in the model. The time history of the impulse load is the same one shown in Figure 2. However, the amplitude of the impulse load was adjusted depending on the dynamic response of the mass points in the model. The amplitudes are proportional to the maximum acceleration responses of the mass points from seismic analysis of the model. The scale factors for impulse load for each of the mass points are shown in Figure 18. The scale factors replicate the similar mode of vibration that is consistent with the maximum response of the mass point at Elev. 57 ft. The impulse response function for the mass point at Elev. 57 ft is shown in Figure 23. The response decays with a rate showing a system damping of nearly 20%.

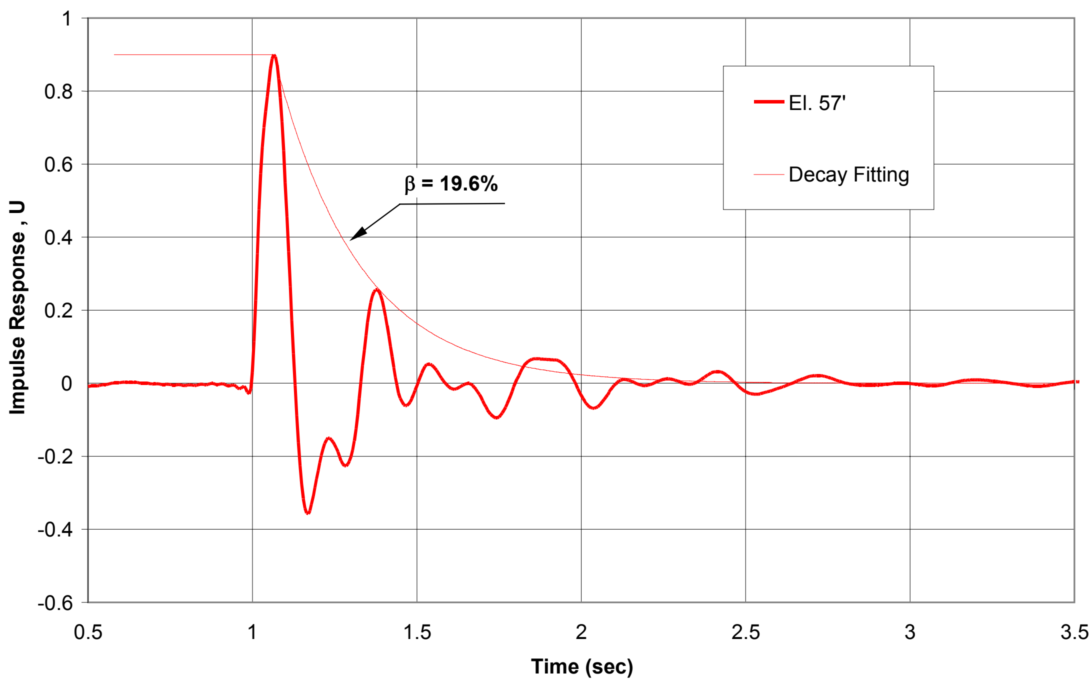


Figure 23. Impulse response function for the node at Elevation 58 ft (SASSI)

To verify the accuracy of the system damping, the analysis of the structure was repeated using the GT-Strudl computer program (Georgia Tech. 2000). The GT-Strudl model is a beam stick model as shown in Figure 24. The SASSI model and the GtStrudl model have the same dynamic fixed base properties. The GT-Strudl model includes the stiffness and dashpots constants in the horizontal and rocking directions at the base of the model. The constants are the average values over a limited frequency range obtained from SASSI impedance analysis (see Figures 20 and 21). The material damping in the structure was modeled by Raleigh damping. The Rayleigh damping was constrained to 5% at 3 and 20 Hz to cover the frequencies of the structure. The Rayleigh damping variation with frequency is shown in Figure 25. The model was subjected to impulse load with the same time history and amplitude variation as used in the SASSI analysis. The GT-Strudl analysis was performed in time domain using the time integration method. The impulse response function at Elev. 57 ft is shown in Figure 26. The decay rate amounts also to about 20% system damping. This confirms the system damping by the SASSI solution is in good agreement with the damping obtained from the GT-Strudl solution as long as the foundation parameters are similar.

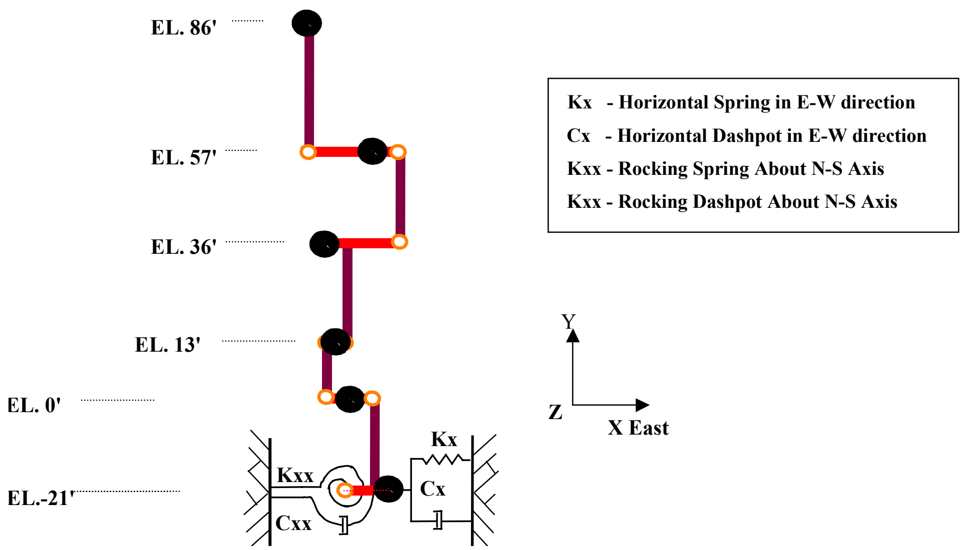


Figure 24. GtStrudl Model with lumped spring and dashpot

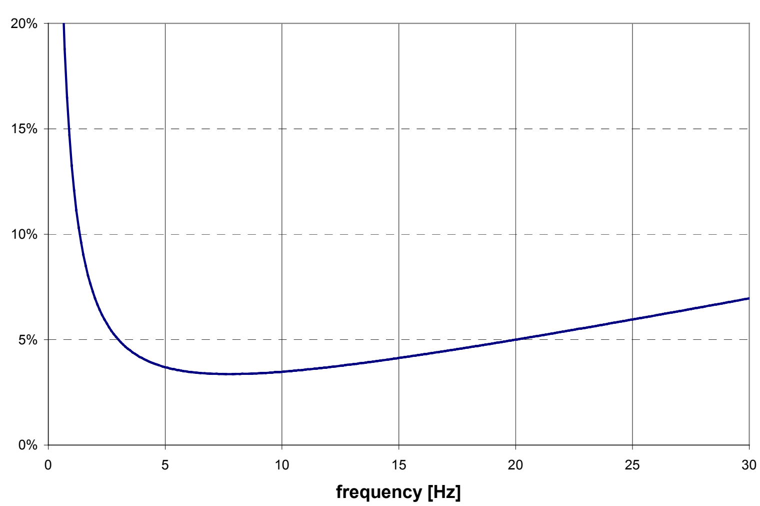


Figure 25. Rayleigh damping used in GtStrudl analysis

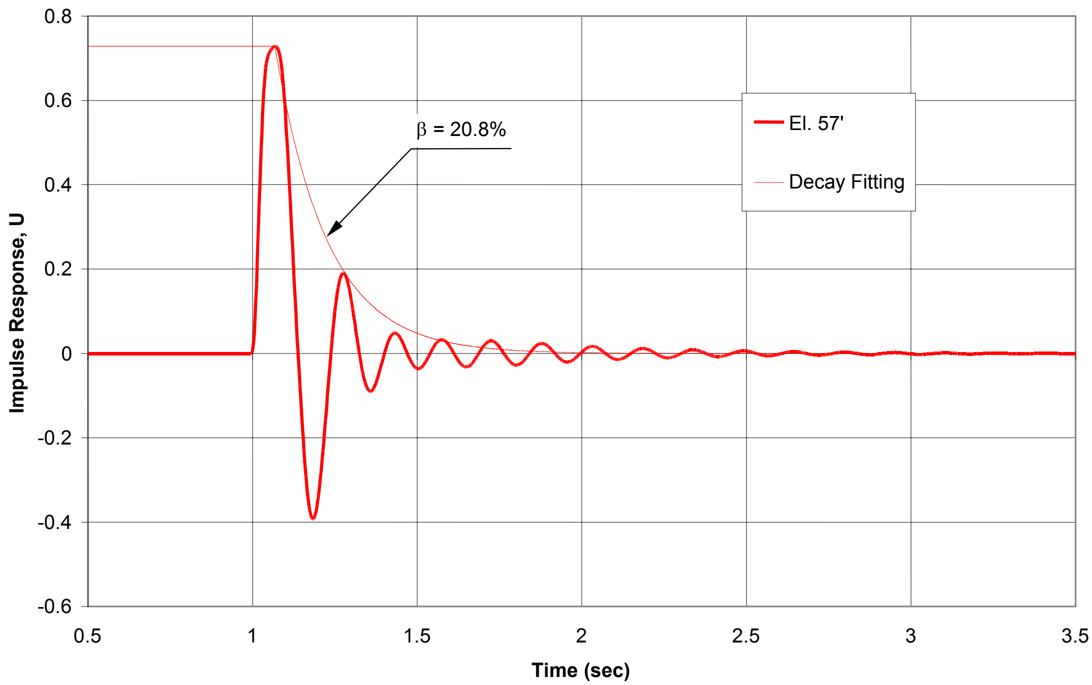


Figure 26. Impulse response function for the node at Elevation 58 ft (GtStrudl)

To compare the seismic response of the structure, both models were analyzed using the acceleration time history of design motion as input. The results in terms of acceleration response spectra at Elev. 57 ft are compared in Figure 27. As shown in this figure, the input motion amplifies in the structure significantly yet a reasonably good agreement can be obtained between the two solutions.

This case study also shows that by applying an impulse load on a SSI system and developing the impulse response function one can effectively obtain a realistic estimate of the total system damping.

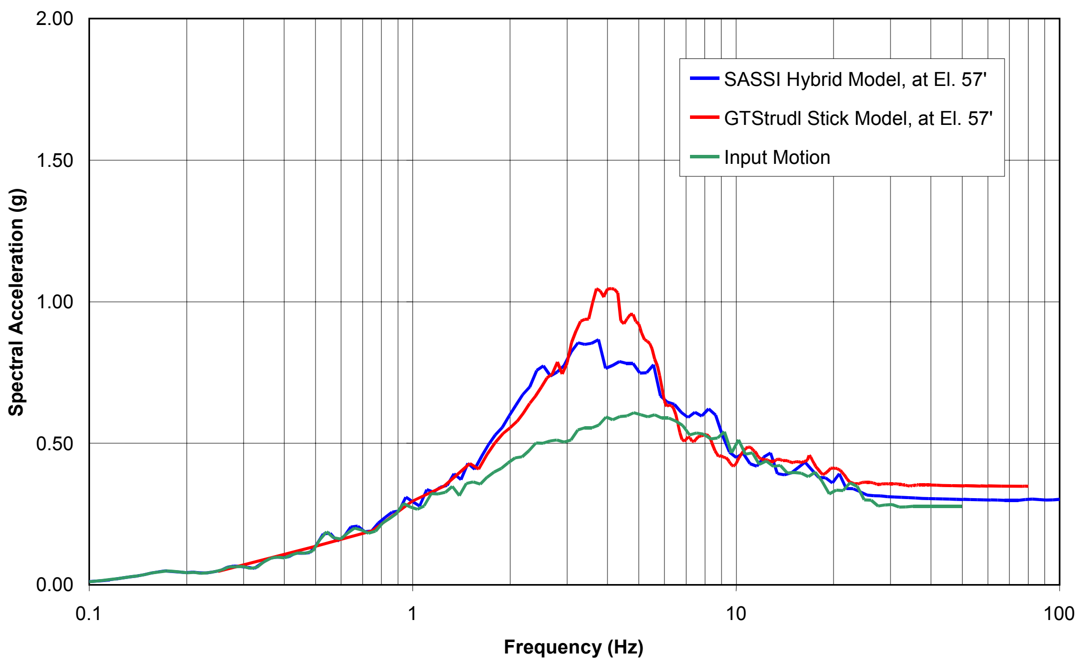


Figure 27. Comparison of acceleration response spectra at Elevation 58 ft

SUMMARY

The simple methods currently available to estimate system damping from dynamic structural responses often fail to predict reasonable results for soil-structure systems due to frequency dependency of the foundation stiffness and dashpot parameters and the complex participation of the SSI and structural modes of vibration in the total response. These methods include the half-bandwidth method, the inverse of the peak and the damping ratio method. In this paper it has been shown that the response from an impulse load applied to the SSI model yields an accurate estimate of system damping while including the effects of material damping, radiation damping as well as composite effects of numerous structural and SSI modes to the dynamic response of the interest. The damping computed may be used to evaluate SSI effects and for input for other types of analysis such as nonlinear time history analysis.

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