RADIATION DAMPING OBSERVED FROM SEISMIC RESPONSES OF BUILDINGS

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SUMMARY

Determination of damping for dynamic response analyses and evaluations of structures is of critical importance in assessing their performances during future events. In most cases, when dynamic analyses are performed, critical damping percentages are adopted from rounded empirical values which represent only structural damping. However, radiation damping, as well as other types of damping, can contribute significantly to the overall effective damping. Two cases of regular buildings that exhibit radiation damping are presented. The features of these two buildings and their site conditions are discussed in detail. The method used to extract damping percentages is discussed. Simple methods are used to confirm the radiation damping percentages. The characteristics of the site, foundation and superstructure of the two buildings are used to show that the radiation damping for such buildings can be substantial and beneficial in assessment of their responses to large earthquakes. If radiation damping is significant, it could be incorporated into the design/analysis procedures.

INTRODUCTION

The purpose of this paper is to draw attention to radiation damping that occurs not only with critical facilities such as containment structures of nuclear power plants but also with regular structures. Recently, through analyses of some of the recorded seismic responses of instrumented building structures, it has been possible to verify and quantify the occurrence of radiation damping [Çelebi, 1997].

Recent codes [ATC3-06, 1978; NEHRP, 1994] incorporate procedures [Veletsos, 1977] for establishing structural and site parameters to determine (i) when radiation damping occurs and (ii) how to quantify it. The Commentary to the NEHRP Recommended Provisions for Seismic Regulations for New Buildings [NEHRP, 1994] states that the foundation damping (ξf) incorporates the effects of energy dissipation in the soil due to:

(a) radiation of waves away from the foundation (radiation or geometric damping),
(b) hysteretic or inelastic action on the soil (soil material damping).

It also states that radiation damping depends on the (1) geometry of the foundation-soil contact area, (2) properties of the structure, and (3) properties of the underlying soil deposits.

The first necessary condition for radiation damping to occur is for the fundamental period of soil, Tsoil=4Hs/vs (where Hs is the depth of the soil layer and vs is the shear wave velocity of the underlying soil) to be $\geq$ the effective fundamental period of the structure, Tstr. Most researchers use the dimensionless frequency (ao= ωoro/vs) as the basic parameter [Gazetas, 1991, Roesset, 1980, Luco, 1980, Todorovska and Trifunac, 1992, Todorovska 1992, Wolf, 1985, 1988, Wolf and Song, 1996, Veletsos, 1977] to show the variation of impedence functions and damping for a structure with a fixed base frequency (ωo), equivalent circular radius of the foundation (ro) [or some length associated with the structure (e.g. height, h) or foundation (e.g. width, D or length, L)] and the shear wave velocity (vs) of the underlying soil layer. A variety of coefficients for

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determining impedance functions and damping coefficients for a wide range of foundation conditions are compiled in a handbook [Sieffert and Cevaer, 1991]. Impedence functions and damping coefficients have been also presented in terms of stiffness ratio ($sr = \omega_o h/v_s$, where h is the height of a structure), slenderness ratio ($hr = h/r_o$), and mass ratio ($mr = m/[\rho r_o^3]$, where m is the mass of the structure and ρ is the mass density of the soil) [Wolf, 1985, 1988, Kramer, 1996]. The equation, $\xi_{eff} = \xi_f + \xi_{str}/(T_{eff}/T_o)^3$, which was presented by Veletsos and Nair [1975], Bielak [1975], Veletsos [1977], and was incorporated into ATC3-06 [1977] and now NEHRP [1994], shows how the effective damping that includes foundation damping and structural damping is beneficial in estimating reduced but realistic structural response. Additional discussions of radiation damping are provided in in ATC3-06 [1978] , Veletsos [1977], Luco [1980], Roesset [1980], Dobry and Gazetas [1985], Todorovska [1992] and Wolf and Song [1996].

The scope of this paper is to discuss radiation damping inferred from recorded responses of two buildings during recent earthquakes. Simple methods are used to confirm the radiation damping percentages. The characteristics of the site, foundation and superstructure of the two buildings are used to show that the radiation damping for such buildings can be substantial and beneficial in their responses to large earthquakes. If this effect does occur and is significant, it should be incorporated into the design/analysis procedures for more realistic response and performance predictions.

FACTS ON THE TWO CASE BUILDING

Case A: Olive View Hospital (OVH), Sylmar, CA.

The new OVH building replaced the original one that was severely damaged during the Ms=6.4 San Fernando (California) earthquake of February 9, 1971 and was later razed. The replacement OVH building was therefore designed in 1976 to increased levels of seismic forces as a reaction to the disastrous fate of its predecessor. The six story OVH building has a very stiff lateral force resisting consisting of concrete and steel plate shear walls. The ground floor and second floor are rectangular and contain the concrete shear walls, typically 10 inches thick and extending along several column lines in both axes of the building. The plan of the building changes to a cross shape (making a four story cruciform tower) starting at the third level. From this level up, 3/8" and 5/8" thick steel shear walls surround the perimeter of the cross. The steel plates between each column line have been stiffened with 3 lines of channel stiffeners on both sides of the plane of the wall [Troy and Richard, 1988]. The foundation of the building consists of spread footings/grade mat on the ground floor. Figure 1 shows a general schematic of the building and seismic sensor locations on floor levels as well as the associated free-field.

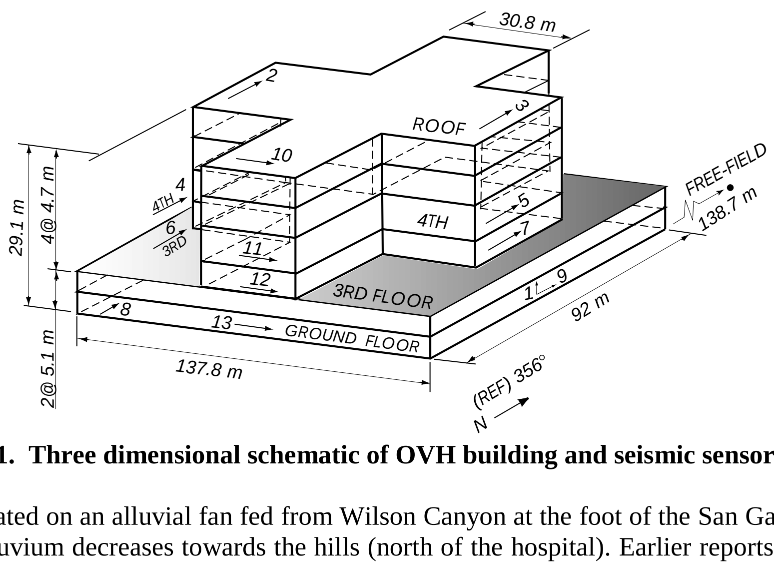

Figure 1. Three dimensional schematic of OVH building and seismic sensor locations

The building is located on an alluvial fan fed from Wilson Canyon at the foot of the San Gabriel Mountains. The thickness of the alluvium decreases towards the hills (north of the hospital). Earlier reports indicate the depth to bedrock to be in the range of 52-92 m (170-300') [Mahin, Bertero, Chopra and Collins, 1976]. There are several borehole logs up to 100 m (~300 feet) taken in the immediate vicinity of the building [Mahin, Bertero, Chopra and Collins, 1976, Fumal, Gibbs and Roth, 1982, and Tinsley, personal communication, 1995]. Typical log descriptions indicate the site has approximately 3-5 m of dense to very dense silty sand and gravel, with underlying alluvial deposits of dense to very dense sand, and of silty sand with gravel and cobbles.

Transfer functions for three possible site profiles corresponding to two different depths and shear velocity pairs for layered substrata are shown in Figure 2. These functions were calculated using software developed by Mueller [personal communication, 1990] and based on shear wave propagation method of Haskell [1953, 1960]. The figure shows that the fundamental site frequency (period) is between 2-3 Hz (0.33-0.5 s) for all models.

Table 1 compares the peak accelerations from two significant sets of response records retrieved from the OVH building and its free-field site. The set of response records retrieved during the (Ms=6.8) Northridge (California) earthquake of January 17, 1994 (at 16 km epicenteral distance) exhibit possibly the largest recorded peak accelerations of the input motions (free-field, 0.91 g and ground level, 0.82 g) and the roof level (2.31g) of a building (Shakal, et al. 1994). The other set of response records is from the (Ms=5.9) Whittier earthquake of October 1, 1987 at an epicenteral distance of 45 km (Huang, et al., 1989). For the Whittier earthquake, the recorded accelerations were much smaller and at an appropriate level to accept the calculated fundamental frequency of OVH as that with little or no SSI [Çelebi, 1997].

Table 1. Peak accelerations and System Identification Results for OVH and PPP

	CASE A : OVH				CASE B: PPP			
Peak Accelerations (A[g])								
Event	Northridge Eq. (1994)		Whittier Eq. (1987)		Loma Prieta Eq. (1989)		Low-Amp. Tests	
	NS	EW	NS	EW	NS	EW	NS	EW
Roof	2.31	0.79	0.20	0.16	0.24	0.38	<0.01	<0.01
Gr. Fl.	0.82	0.42	0.06	0.06	0.17	0.21	<0.01	<0.01
FF	0.91	0.61	0.06	0.05	0.21	0.26	-	-
System Identification								
Event	Northridge Eq. (1994)		Whittier Eq. (1987)		Loma Prieta Eq. (1989)		Low-Amp. Tests	
f_o (Hz)	2.5	2.5	3.33	3.33	0.38	0.38	0.48-0.57	
T_o (s)	0.4	0.4	0.3	0.3	2.63	2.63	1.75-2.08	
ξ(%)	10-15	5-10	1-4	5-8	11.6	15.5	0.6-3.4	

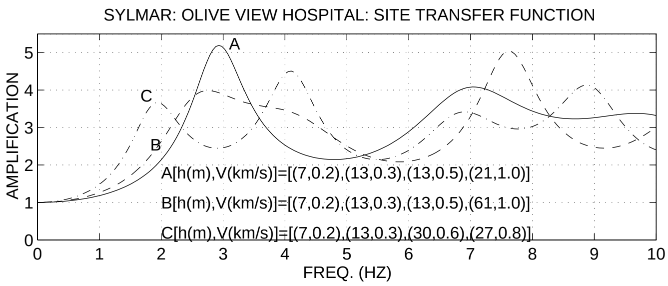

Figure 2. Site transfer functions of OVH building site (the parameters of profiles A, B and C are as follows: h – the layer thickness, and V – shear wave velocity).

The design of OVH building was based on two levels of expected performance of the building represented by the design-level and survivability level earthquakes with zero period acceleration (ZPA) of 0.52 g and 0.69 g, respectively. The survivability earthquake is postulated for a Richter magnitude 8.5 earthquake at 25 km distance. The two design response spectra (for 5% damping) are compared in Figure 3 with response spectra of the free-field at the OVH grounds. As seen in this figure, OVH was tested beyond its design limits during the 1994 Northridge earthquake (Ms=6.8) due to large peak accelerations and spectral characteristics of the recorded motions.

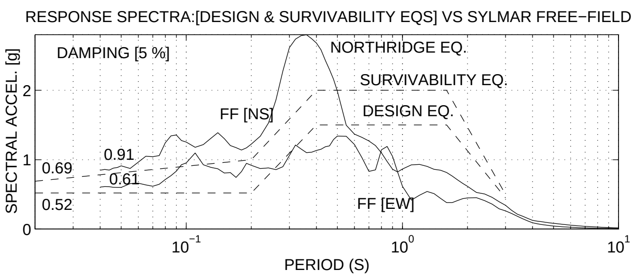

Figure 3. Comparison of design response spectra with response spectra of free-field motions recorded at the OVH site during the Northridge earthquake.

2.2. Case B: Pacific Park Plaza (PPP), Emeryville, CA.

The 30-story PPP building was constructed in 1983 and is the tallest reinforced concrete building in northern California. It is an equally spaced, three-winged, cast-in-place, ductile, moment-resistant reinforced concrete framed structure. A plan view, a three-dimensional schematic, and the instrumentation scheme of the building including the north and south (SFF) free-field stations, is shown in figure 4 [Çelebi, 1992, 1996]. The foundation is a 5-foot-thick concrete mat supported by 828 (14-inch-square and 20-25 m in length) prestressed concrete friction piles in a primarily soft-soil environment (average shear-wave velocity ~ 250 - 300 m/s and a depth of approximately 150 m to harder soil) [Hensolt and Brabb, 1990]. Using Haskell's procedure as in Case A, a site frequency of 0.7 Hz is obtained.

Two sets of motions (summarized in Table 1) are used to characterize the response of PPP - one obtained during the October 17, 1989 (M_s=7.1) Loma Prieta earthquake [LPE] (at epicentral distance of ~100 km) and the other from low-amplitude tests conducted following LPE. During the LPE, PPP was not damaged, even though it was subjected to considerably amplified motions. Figure 5 shows the east-west components of acceleration recorded at the roof and ground floor of the structure, at the associated free-field station (SFF in fig. 4*)*, and also at Yerba Buena Island (YBI), the closest rock site(also at epicentral distance of ~ 100 km). Both the amplitudes of peak accelerations (0.26 *g* for SFF and 0.06 *g* for YBI) and the response spectra imply that the motions at SFF were amplified by as much as five times when compared with YBI. Furthermore, the difference between the peak accelerations at SFF (0.26 *g*) and the ground floor (0.21 *g*) infer the possibility of significant SSI.

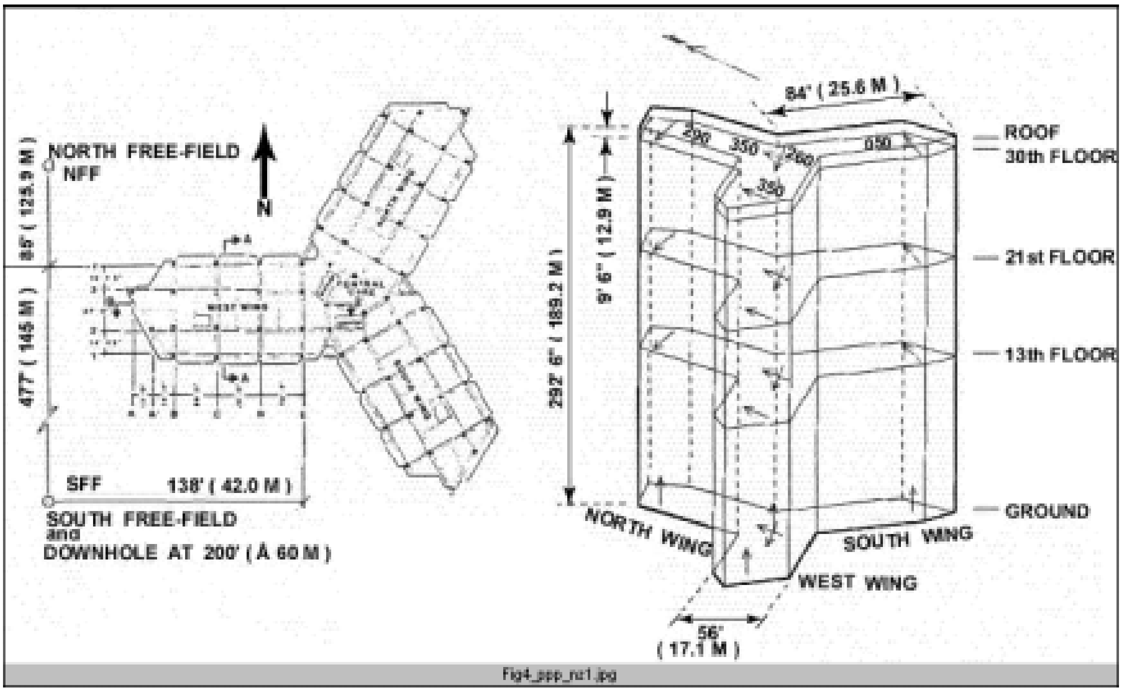

Figure 4. Plan View, three dimensional schematic and instrumentation scheme of PPP

The strong-motion response of the PPP has been studied in detail by Çelebi and Safak [1992], Anderson and Bertero [1994], Anderson and others [1992], Bertero and others [1992], Kagawa and others [1993], Aktan and others [1992] and Kambhatla and others [1992]. The predominant response modes of the building and the associated frequencies (periods) [0.38 Hz (2.63 s), 0.95 Hz (1.05 s), and 1.95 Hz (0.51 s)] are identified by all these investigators using different methods, including spectral analyses, system identification techniques, and mathematical models. These modes are torsionally-translationally coupled [Çelebi, 1996]. The frequencies are shown in the cross-spectra (S_{xy}) of the orthogonal records obtained from the roof and ground floor, the south free-field site (SFF), and the normalized cross-spectra of the orthogonal records (fig. 6). A site frequency at 0.7 Hz (1.43 s) is also identified. The peak at 0.7 Hz that appears in the cross-spectrum of the roof appears as the dominant peak in the cross-spectra of the ground floor and SFF. When the normalized cross-spectra are calculated for the ground floor and SFF, the site frequency at 0.7 Hz is distinguishable from the structural frequencies in the normalized cross-spectrum of the roof (fig. 6). The dynamic characteristics determined from low-amplitude tests performed after LPE using spectral analyses techniques are also summarized in Table 1 [Marshall and others, 1992; and Çelebi and others, 1993; Çelebi, 1996]. Because of the low signal/noise ratio, system identification techniques could not be applied to the low-amplitude test data.

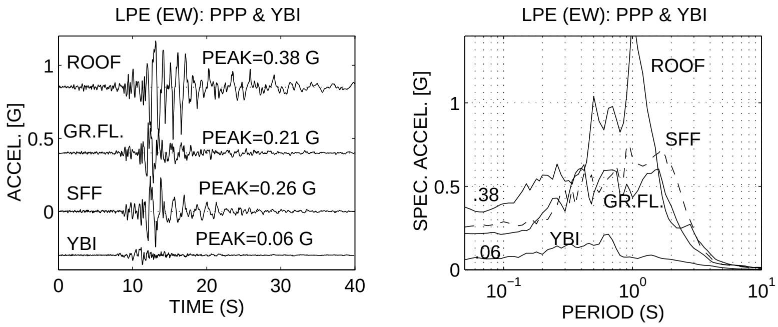

Figure 5. Recorded (EW) accelerations and corresponding response spectra at the free-field, ground floor and roof of Pacific Park Plaza (PPP), and at Yerba Buena Island (YBI) [at approximately the same distance as PPP] depict the level of amplification at PPP site.

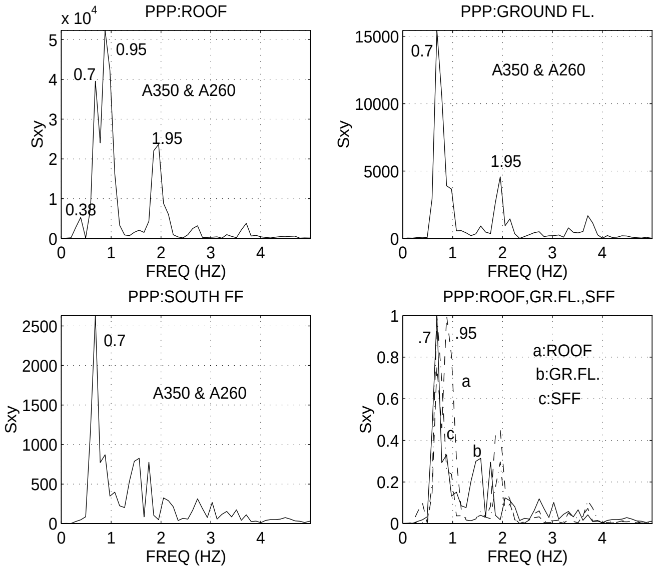

Figure 6. Cross-spectra of orthogonal accelerations (A350 & A260) at the roof, ground floor, free-field of PPP. Also shown (bottom right) is the normalized cross-spectrum depicting structural and site frequency peaks. (350 & 260 depict degrees clockwise from true north).

SYSTEM IDENTIFICATION FOR CASES A AND B

Table 1 also shows results of system identification analyses performed using earthquake acceleration records of both buildings. Nonlinear effects were not incorporated in this effort. In each case, the roof records were the output and the ground floor records were the input. The ARX model based on the least squares method in the public domain program MATLAB was used [Mathworks, 1994]. As previously mentioned, for PPP, spectral analyses techniques were used to identify dynamic characteristics from low-amplitude test data [Marshall and others, 1992, Çelebi, 1996)]

For the OVH building, the damping ratios extracted from system identification procedures are 10-15 % (NS) and 5-10 % (EW) for Northridge and 1-4 % (NS) and 5-8 % (EW) for Whittier. These results are consistent with the relative level and orientation of predominant motion for the earthquakes. For the PPP, the damping ratios extracted from the system identification analyses corresponding to the 0.38-Hz first-mode frequency are 11.6 percent (north-south) and 15.5 percent (east-west) [Çelebi, 1996]. Such unusually high damping ratios extracted from recorded data of these conventionally designed/constructed buildings require explanation. Both buildings, with their large mat foundations in relatively soft geotechnical environments, are capable of energy dissipation in the soil due to radiation and/or material damping.

In Table 2, two approaches for estimating radiation damping of the two buildings are summarized. In the first approach, adopted fixed base fundamental periods are used with the effective periods to estimate foundation damping from Figure 7 (ATC3-06, 1978, Veletsos, 1977, NEHRP, 1994). This approach yields radiation damping of 20 % or more for OVH and 5-8 % for PPP. The second approach, following Gazetas (1991), utilizes impedence formulas to calculate radiation damping. This approach is valid when the dimensionless frequency, ao= ωoro/vs >1. As seen in Table 2, for OVH, this approach works and yields radiation damping of 29 % for the adopted parameters summarized in the table. For PPP, since ao <1, calculations were not made. However, it should be stated herein that the setting for PPP is such that with all the piles, the effective foundation radius is possibly much higher than calculated and that the shear velocity for the top layers is possibly smaller than 250m/s. In any case, both approaches show that for OVH, there is strong evidence of radiation damping. For PPP, the evidence suggests that radiation damping is likely but more observations are needed to confirm it.

Table 2. Two approaches in calculation of foundation damping

	OVH	PPP
f o(Hz) [Fixed]	3.33	0.48-0.57
feffective	2.5	0.38
To (s) Fixed]	0.3	1.75-2.08
Teffective	0.4	2.63
fsite (Hz) [Tsite, (S)]	2Hz [0.5s]	0.7 HZ, [1.33s]
A (width B[m] .length L[m])	12678= (137.8mx92m)	1600=40mx40m
ro= √(A/π)	63.5	22.6
h (m)	29.1	89.2
I. Approach in ATC3-06 (1978), Veletsos (1977), NEHRP (1994)		
He=0.7(h/r)	0.46x0.7=0.32	3.95x0.7=2.77
Teffective/To	1.33	1.26-1.50
Foundation Damping [ξ] (%)	~20-25	5-8
II. Approach by Gazetas (1991) based on dimensionless frequency (if ao>1)		
Shear wave velociy vs (m/s)	250	250
Dimensionless freq. ao= ωoro/vs	5.31	0.27-0.32 < 1
ρ [Mg/m3]	1.6	1.6
ν [Poisson's Ratio]	0.4	0.4
G=ρ vs2 [Mpa]	100	100
C=Aρ vs [kN.s.m-1] [103]	5071	640
Ky = 8Gr/(2-ν) [kN/m] [106]	31.75	
m (mass) [kN.s2.m-1] [103]	23.5	
[ξ] = [C /2]/√(Ky m) (%)	29	N/A

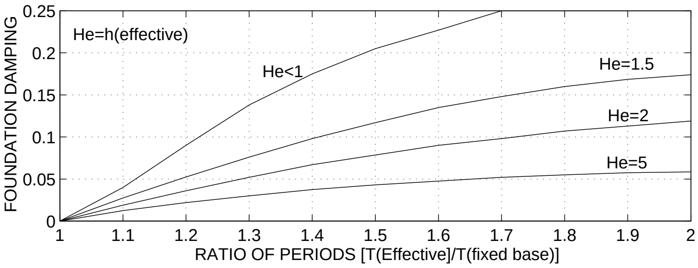

Figure 7. Foundation damping [adopted from ATC3-06 , 1978 and Veletsos , 1977].

CONCLUSIONS

Critical damping percentages adopted during design/analysis phase of structures are from rounded empirical values that almost always represent only structural damping. However, radiation damping, as well as other types of damping, can contribute significantly to the overall effective damping. In this paper, two cases of regular buildings that exhibit radiation damping are presented. The features of these two buildings and their site conditions facilitate application of simple techniques to estimate damping percentages. It is shown that the radiation damping for such buildings can be substantial and beneficial in their responses to large earthquakes. If identified that this effect can occur and is significant, then, it should be incorporated into the design/analysis procedures.

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