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Approximate Earthquake Analysis for Regular Base Isolated Buildings Subjected to Near Fault Ground Motions

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Abstract—**An approximate earthquake modal analysis of base isolated regular buildings (AMA-BI) subjected to near fault ground motions is developed and presented in this paper. The basis of this procedure benefits from two important properties of base isolated structures; the concentrated nonlinearity at the base isolation level and the fact that the base isolation and the superstructure behave as two bodies in contact with discontinuity at the isolation layer. The AMA-BI treats the base as it is; a nonlinear system, and the superstructure as elastic. This latter can be decomposed into subsystems so that when superposed and combined with the response of the base they give an overall behavior and response very close to that obtained when using the nonlinear time history analysis (NLTHA) of the base isolated structure. An example of application is treated to evaluate the accuracy of the procedure.**

Keywords: Base Isolation, Superstructure, Nonlinearity, Modal Analysis, Near Fault

1 Introduction

Now, evident is the performance of structures mounted on an isolation system during earthquakes. The isolation system deflects the seismic energy so that will not be transferred to the superstructure. The benefits gained by using such technique are substantial and can be itemized after Stanton and Roeder (1991) [13] as:

- Reduced floor accelerations and interstory drifts;
- Reduced (or no) damage to structural elements;
- Better protection of buildings' contents;
- Concentration of nonlinear, large deformation behavior into one group of elements (the isolation bearings and dampers).

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The last item highlights an important inherent property of a well selected and designed isolation system. The superstructure may remain elastic under these conditions and most deformations occur at the isolation level, so that the structure's expensive contents and equipments remain intact from minim damages. However, the challenge is how to select the appropriate isolation system parameters. Several design and analysis methods were proposed, but a common problem arise in the complexity of implementation that require an extensive effort to be performed. Therefore, we are interested in developing simple but efficient procedures that lead to an appropriate design with minimum effort.
Based on the above itemized seismic isolation features, an approximate earthquake modal analysis procedure (AMA-BI) is developed and illustrated through examples.

2 Approximate Earthquake Modal Analysis (AMA-BI)

2.1 Uncoupled Equations of Motion

The matrix form of differential equations governing the response of a MDOF base isolated structure (see figure 1) to earthquake induced ground motion are as follows:

$$M\ddot{U} + C\dot{U} + KU = -MR(\ddot{u}_g(t) + \ddot{x}_b) \quad (1)$$

$$R^T M\left[\ddot{U} + R(\ddot{u}_g(t) + \ddot{x}_b)\right] + m_b(\ddot{u}_g(t) + \ddot{x}_b) + f = 0 \quad (2)$$

Eq. 1 corresponds to the superstructure while Eq. 2 corresponds to the base isolation level, where M and K are the mass and lateral stiffness matrices of the superstructure, R is the vector of earthquake influence coefficients. The damping matrix C would not be needed in the approximate modal analysis of earthquake response (AMA-BI); instead modal damping ratios suffice.
The idealization of a base isolated structure as a block mass mounted on isolation system is reasonable since the the peak and overall response will not be altered [10]. Also, the structural period and damping ratios have no noticeable effect on the peak response [9]. For this and as a reasonable approximation we neglect the term $R^T M\ddot{U}$

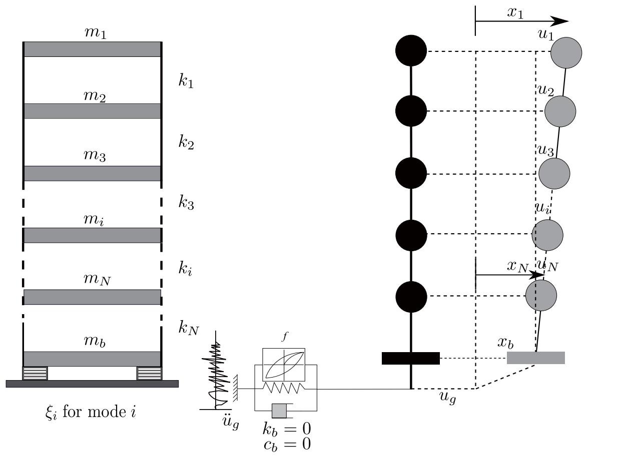

Figure 1: MDOF base isolated structure

from Eq.2, this gives:

$$\ddot{x}_b + \frac{f}{M_{tot}} = -\ddot{u}_g(t) \tag{3}$$

where $M_{tot} = M_s + m_b$; M_s is the total mass of the superstructure and m_b is the base mass.
The procedure consists to solve Eq.3 for $\ddot{x}_b$ that will be added to the ground motion acceleration to constitute a new excitation for the superstructure to determine finally the total response quantities of interest.
Figure 2 illustrates the AMA-BI procedure for MDOF base isolated systems. The MDOF system is equivalent to combination of the response of the total mass M_{tot} mounted on the isolation system subjected to the earthquake acceleration $\ddot{u}_g(t)$ and series of SDF systems pinned at their bases having unit mass and natural periods T_i subjected to the excitation $(\ddot{u}_g(t) + \ddot{x}_b)$. Shown in the same illustration are the equations to be solved for each subsystem and the corresponding vibration properties. The modal matrix for the MDOF base isolated structure is denoted by Φ while that for the superstructure is denoted Φ_{sup}, their shapes are depicted in figure 3. The use of the modal matrix Φ_{sup} ($\Phi_{sup} = \Phi(1:N, 1:N)$) instead of the modal matrix computed using M and K is due to the fact that we have to keep the nodal motion shapes. In addition, the natural frequencies (or natural periods T_i) used for the superstructure ($\Omega^2_{sup} = \Omega^2(2:N+1, 2:N+1)$ are those calculated using the mass matrix of the whole structure and its corresponding stiffness matrix after elimination of the first entry (first period) which is the isolation period, the rest is taken as the periods of the SDF systems constituting the superstructure. The floor displacements relative to the base u_i are computed as a superposition of the modal contributions as follows:

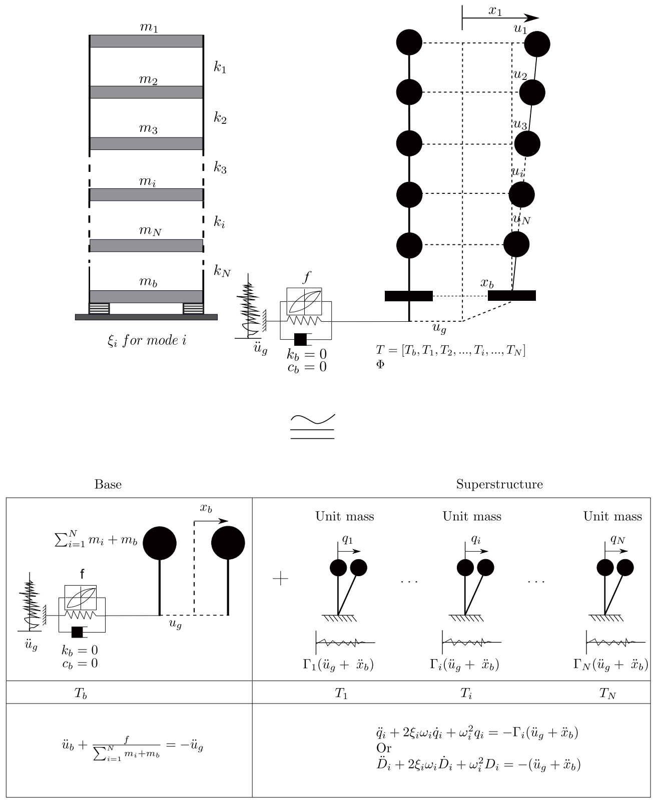

Figure 2: AMA-BI procedure for MDOF base isolated structures

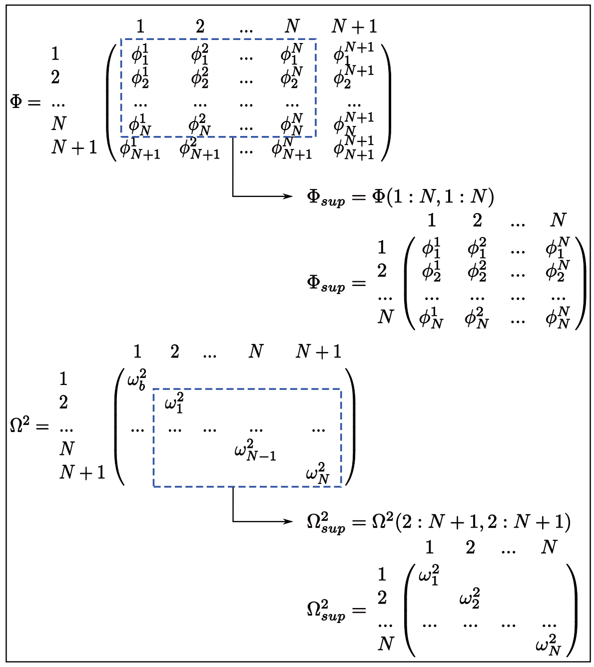

Figure 3: Determination of Φ_{sup} and Ω^2_{sup} the modal and spectral matrices of the superstructure

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$$U = \Phi_{sup} Q \ \text{ where } \ Q = \begin{pmatrix} q_1 \\ q_2 \\ . \\ . \\ . \\ q_N \end{pmatrix} = \begin{pmatrix} \Gamma_1 D_1 \\ \Gamma_2 D_2 \\ . \\ . \\ . \\ \Gamma_N D_N \end{pmatrix}$$

$$\text{and } \ U = \begin{pmatrix} u_1 \\ u_2 \\ . \\ . \\ . \\ u_N \end{pmatrix}$$

Γ_i is the modal participation factor, which is a measure of the degree of to which the i^{th} mode participates in the response of the superstructure, and it is described as follows:

$$\Gamma_i = \frac{\phi_{sup\ i}^T M R}{\phi_{sup\ i}^T M \phi_{sup\ i}}$$

Furthermore, the modal floor displacements relative to the base are combined to compute the total response using the base displacement and the transformation matrix, Ψ:

$$X = \begin{Bmatrix} x_1 \\ x_2 \\ . \\ . \\ . \\ x_N \\ x_b \end{Bmatrix} = \Psi \begin{Bmatrix} u_1 \\ u_2 \\ . \\ . \\ . \\ u_N \\ x_b \end{Bmatrix} \ \text{ where } \ \Psi = \begin{pmatrix} \Phi_{sup} & [1] \\ [0] & 1 \end{pmatrix}$$

The same procedure as done for displacements determination is to be followed for floor accelerations computation:

$$\ddot{U} = \Phi_{sup} \ddot{Q} \ \text{ where } \ \ddot{Q} = \begin{pmatrix} \ddot{q}_1 \\ \ddot{q}_2 \\ . \\ . \\ . \\ \ddot{q}_N \end{pmatrix} = \begin{pmatrix} \Gamma_1 \ddot{D}_1 \\ \Gamma_2 \ddot{D}_2 \\ . \\ . \\ . \\ \Gamma_N \ddot{D}_N \end{pmatrix}$$

$$\text{and } \ \ddot{U} = \begin{pmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ . \\ . \\ . \\ \ddot{u}_N \end{pmatrix}$$

Therefore the floor accelerations relative to the ground are then computed as a superposition of the above calculated quantities:

$$\ddot{X} = \begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \\ . \\ . \\ . \\ \ddot{x}_N \\ \ddot{x}_b \end{Bmatrix} = \Psi \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ . \\ . \\ . \\ \ddot{u}_N \\ \ddot{x}_b \end{Bmatrix}$$

Forces in each floor are computed from the total floor accelerations as follows:

$$F = M_T \ddot{X}_{tot}; \ \ \ddot{X}_{tot} = R\ddot{u}_g(t) + \ddot{x} \ \text{ and } \ R = (1\ 1\ ...\ 1)^T$$

where M_T is the mass matrix of the structure including the base.
It should be noted that in the solution of Eq.3 the total mass of the structure is to be adjusted so that the isolation period resulted is equal to the first period determined from the solution of the eigenproblem characterizing the whole structure. For 2DOF base isolated structures (one story + base level) the transformation matrix is different from that given for MDOF systems and is described as follows:

$$\Psi = \begin{pmatrix} \frac{1}{M_{tot}} & 1 \\ 0 & 1 \end{pmatrix}$$

2.2 Six-Story Structure with Lead-Rubber Bearing Isolation System

The analysis of a six-story reinforced concrete base isolated structure with LRB isolation system is considered. The 3D, plan and elevation views are shown in figure 4. The reinforced concrete superstructure is designed to resist lateral loads using column elements. The vertical axis of centers of mass is offset from the geometric center of the structure from inducing a mass eccentricity of 17 cm in the Y direction. The uncoupled translational period of the superstructure $T_s = 0.64\ sec.$ in both X and Y directions for a complete three-dimensional representation and $T_s = 0.39\ sec.$ in both X and Y directions for a three-dimensional shear representation assuming rigid floors. Damping of 5% of critical is used for the superstructure in all the modes.
The LRB isolation system designed based on practical parameters. The average isolation yield strength Q_y is set $5\%W$, where W is the total weight of the structure $W = 13560.47\ kN$. The yield displacement u_y is set to 1 cm. The postyield stiffness is determined based on an isolation period T_b of 2 sec.

Eight records are selected from the CDMG (California Division of Mines and Geology, Sacramento, USA) suite of ground motions representing near fault effects, low and large ground velocities which are specifically suggested by the CDMG for design of seismically isolated structures [6]. The records and their characteristics are summarized in table 1. The superstructure is modeled as a three-dimensional shear structure. Lead-Rubber Bearings are modeled using biaxial model for elastomeric bearings to capture the interaction and keeping its effect on the seismic response. The dynamic response is computed for the selected accelerograms, the structure is subjected to each component in the X direction only, but keeping the biaxial interaction so that the response will be affected. In the application of AMA-BI procedure the superstructure was simulated as lumped mass model (stick model) with condensed stiffness at each story calculated

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Table 1: Selected components of CDMG suite of earthquakes

Earthquake	PGA (g)	PGV ($cm/sec.$)	PGD (cm)
El Centro 1979 (Array#6 station) 230^o	0.436	108.709	55.165
Loma Prieta 1989 (Hollister station) 90^o	0.178	30.891	20.418
Loma Prieta 1989 (Lexington Dam station)90^o	0.409	94.982	25.814
Landers 1992 (Lucerne valley station) Long.	0.703	25.718	8.824
Northridge 1994 (Newhall station) 90^o	0.583	74.841	17.595
Petrolia 1992 (Petrolia station) 90^o	0.662	89.454	30.577
Northridge 1994 (Sylmar station) 90^o	0.604	76.936	15.217
Landers 1992 (Yermo station) 360^o	0.151	29.032	22.779

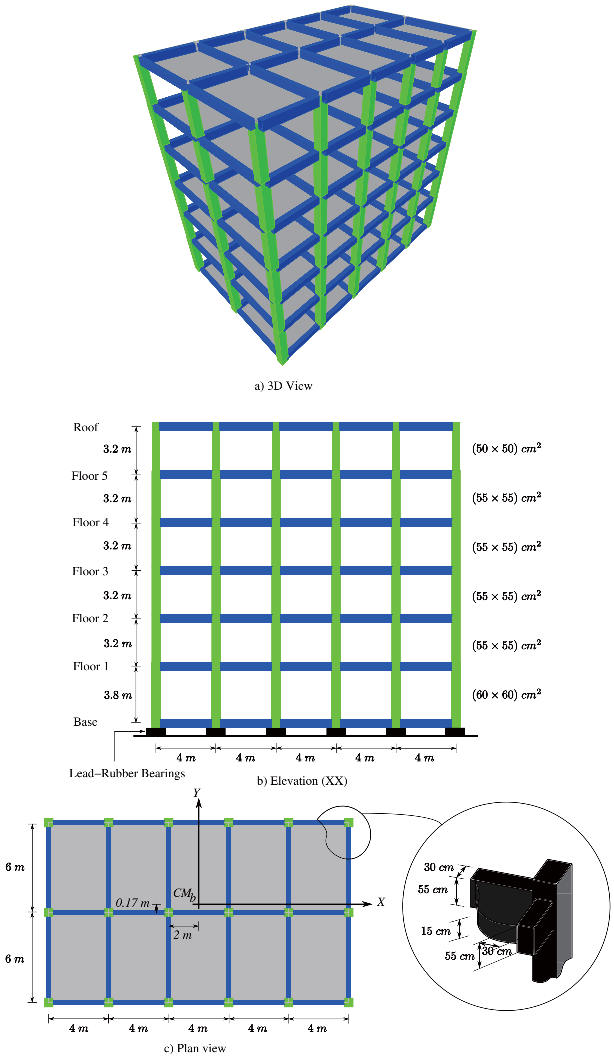

Figure 4: Six-story reinforced concrete structure on lead-rubber bearing isolation system

by summation of column stiffnesses in the X direction. The AMA-BI implies introduction of mass matrix, modal and spectral matrices in addition to the rest of inherent inputs (see section 2.1), they are given in table 2 (units: kN, m, s).
To test the AMA-BI procedure, the response of the six-story reinforced base isolated structure (figure 4) to the eight components (table 1), was determined using two methods: (1) rigorous nonlinear time history analysis by solving the governing coupled equations of motion; and (2) the AMA-BI procedure. Such a comparison for roof displacement, roof and base accelerations, roof and base forces and base shear force is presented in figures 5 to 12. The peak response values due to the induced ground motions in the X direction are shown in tables 3 and 4 with the corresponding errors in the AMA-BI.

2.3 Discussion

The AMA procedure which is based on uncoupling the equations of motion gave results very close to that obtained when using NLTHA with negligible errors that even could happen to any comparison between two computer programs that use the rigorous NLTHA (e.g. ETABS and 3D-BAIS). Also, the AMA predicted accurately the shape of peak floor forces distribution throughout the hight of the structure.
The large errors in floor forces occurred when the structure is subjected to the Loma Prieta, Hollister 90^o component, but still under 5% which can also be neglected, but lets interpret it: the isolation system works better under sever ground motions with large peak velocities; the yield strength achieves its higher value under higher velocity ([7],[8],[10]), if an earthquake has lower peak velocity (which is the case of Loma Prieta, Hollister 90^o) the isolation system do not perform properly as should be, the indirect result in this case is that the AMA-BI predicts peak responses with larger errors since the approximate procedure was developed mainly benefiting from the isolation property.
The accuracy of AMA-BI was proven using the above structure, however, its efficiency is demonstrated by the fact that the time spent for complete analysis of the structure subjected to an earthquake component with 4000 points using AMA-BI procedure is about 3 seconds, while

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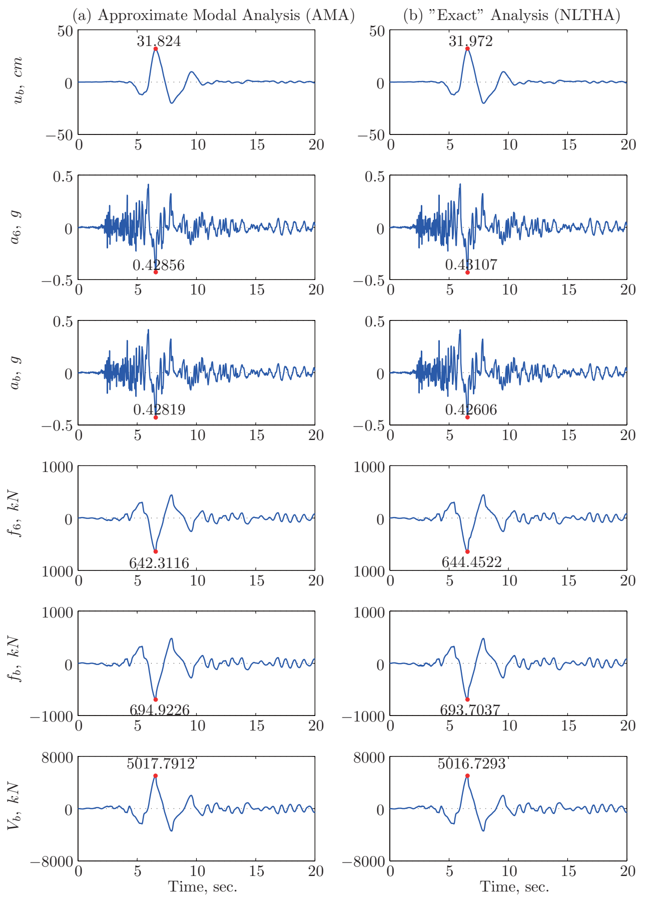

Figure 5: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to El Centro 1979, Array#6

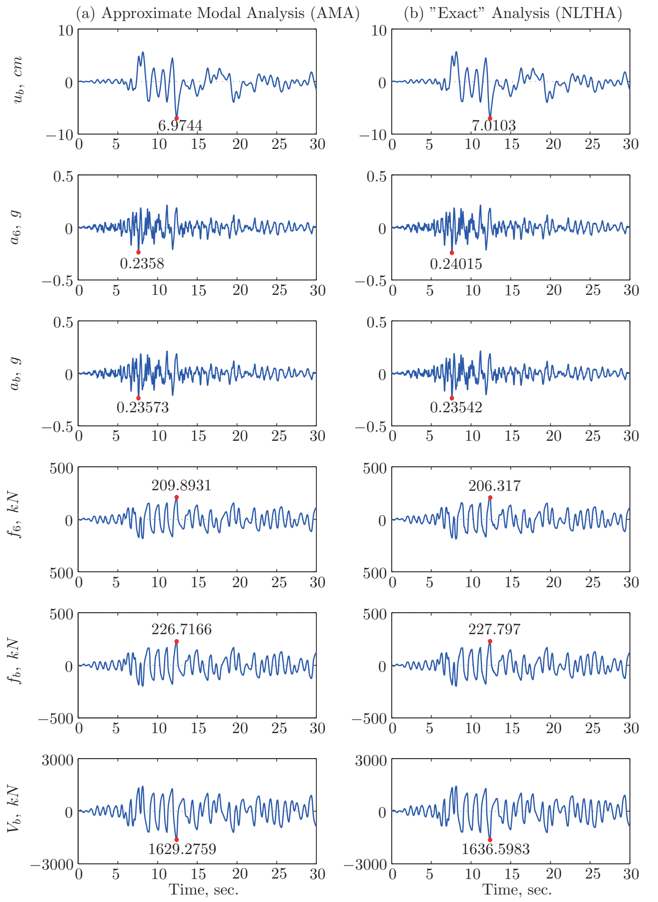

Figure 6: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Loma Prieta, Hollister 90^o

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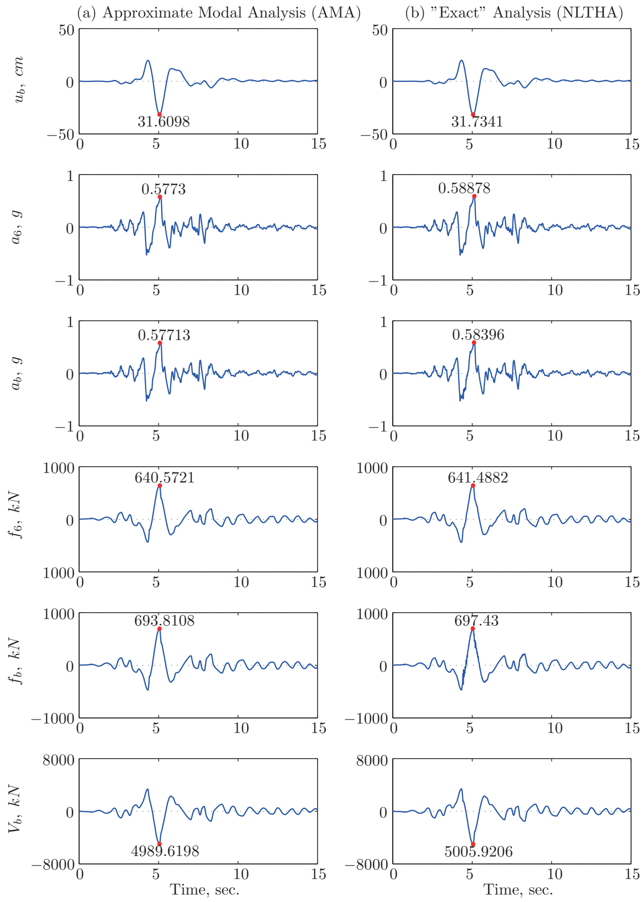

Figure 7: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Loma Prieta, Lexington 90^o

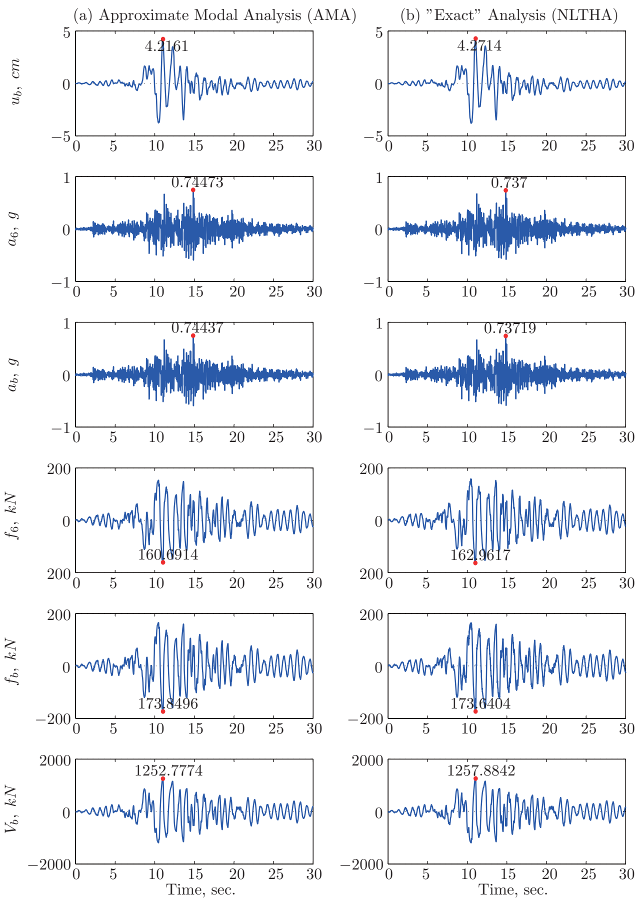

Figure 8: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Landers, Lucerne *Long*.

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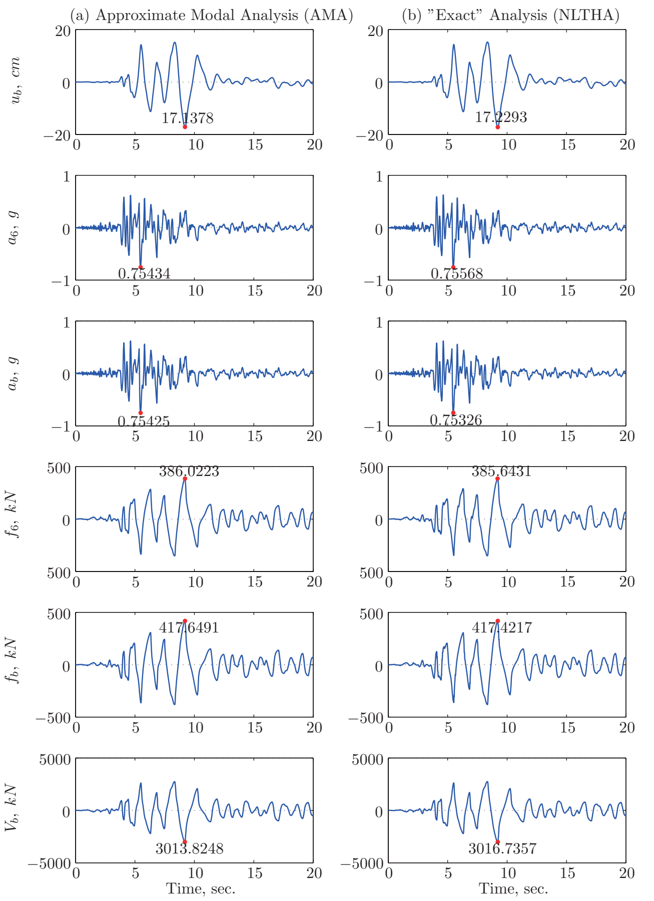

Figure 9: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Northridge, Newhall 90^o

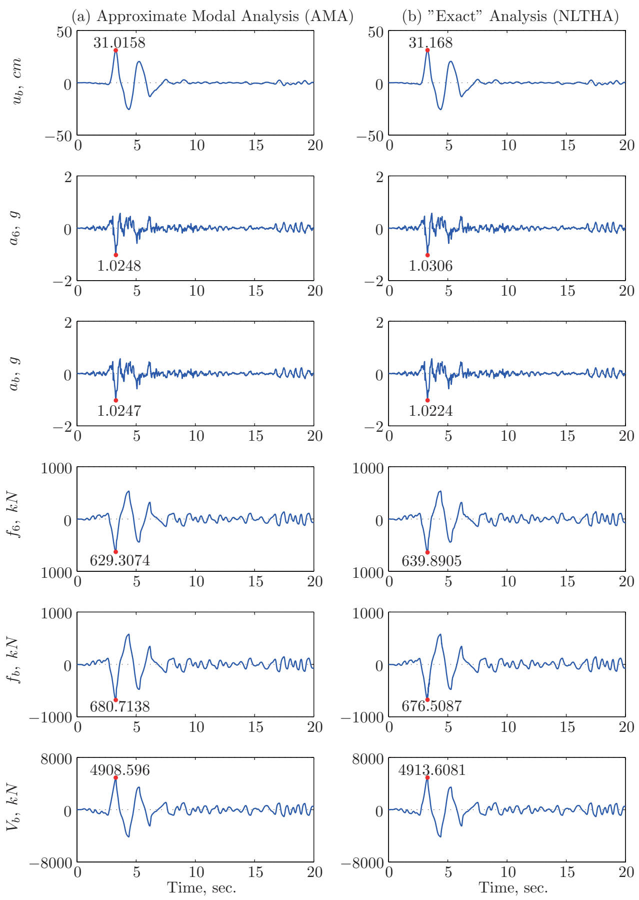

Figure 10: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Petrolia, Petrolia 90^o

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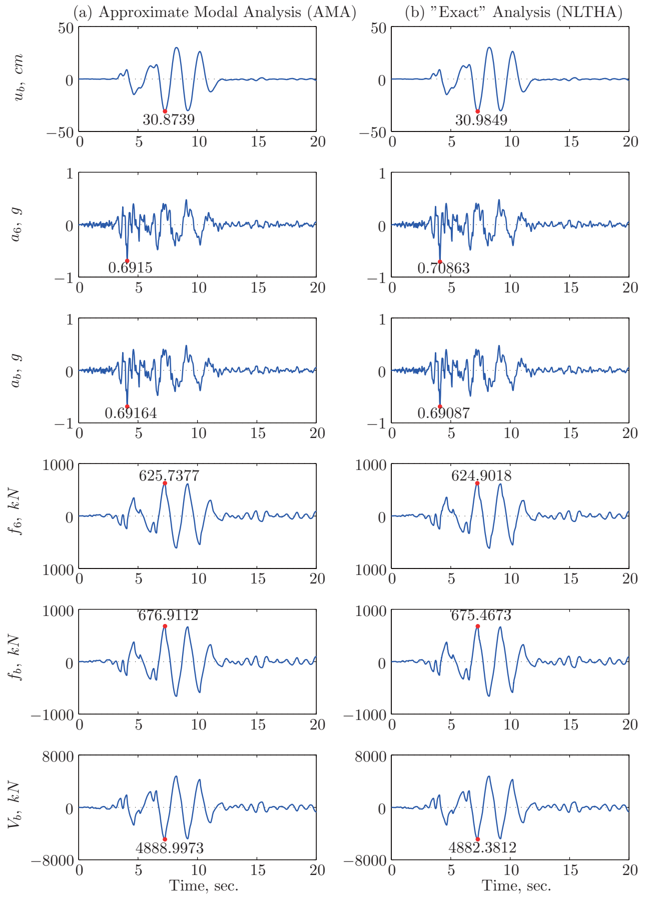

Figure 11: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Northridge, Sylmar 90^o

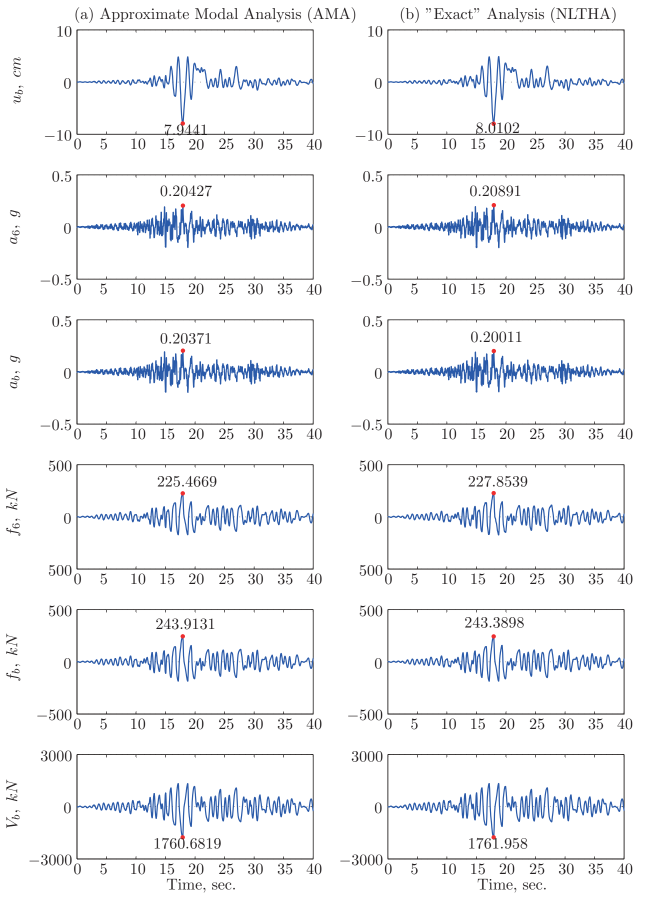

Figure 12: Comparison of response histories obtained by NLTHA and AMA-BI for the six-story structure with LRB subjected to Landers, Yermo 360^o

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the application of the NLTHA takes more than 180 seconds.
Also, the reader may be referred to the paper [12] where has been presented a response spectrum analysis procedure (RSA-BI) which constitutes the particular case when the AMA-BI is specialized for peak responses. This procedure is based on a response spectrum developed for base isolated buildings (were called in the paper "SIRS" or Seismic Isolation Response Spectrum). The idea of construction of the SIRS was initiated by Zayas et *al* [14] for friction pendulum system, after; Ryan and Chopra [10] have developed the theory to include other systems and structures. The RSA-BI was found to give consistent peak response values when compared against results of the NLTHA, hence, it can replace the existent theory of linearization that uses the effective stiffness and effective damping.

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Table 2: Eigenvalues and Eigenvectors of the structure

Mode(ω^2)	Story(mass $kN.s^2/m$)	ϕ
1(9.874)	6 (176.59)	0.02696
	5 (197.61)	0.02696
	4 (201.39)	0.02695
	3 (201.39)	0.02694
	2 (201.39)	0.02692
	1 (209.62)	0.02690
	B (191.083)	0.02688
2(13579.259)	6	0.04034
	5	0.02976
	4	0.01656
	3	-0.00002
	2	-0.01659
	1	-0.02978
	B	-0.03552
3(49129.860)	6	0.04222
	5	0.00215
	4	-0.02678
	3	-0.03591
	2	-0.01849
	1	0.01260
	B	0.03054
4(98243.604)	6	0.03280
	5	-0.02945
	4	-0.02924
	3	0.01420
	2	0.03665
	1	0.00491
	B	-0.02804
5(158261.570)	6	0.01866
	5	-0.03840
	4	0.01236
	3	0.03368
	2	-0.02522
	1	-0.02406
	B	0.02692
6(215497.986)	6	0.00936
	5	-0.02962
	4	0.03801
	3	-0.01763
	2	-0.01610
	1	0.03764
	B	-0.02383
7(253309.931)	6	0.00385
	5	-0.01499
	4	0.02820
	3	-0.03611
	2	0.03722
	1	-0.03133
	B	0.01542

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Table 3: Peak response results of AMA-BI compared with NLTHA

Quake	Story	Displacement (cm)			Acceleration (g)			Force (kN)			Shear (kN)		
		AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)
Array#6	6	31.82	31.97	-0.46	0.43	0.43	-0.58	642.31	644.45	-0.33	642.31	644.45	-0.33
	5	31.82	31.95	-0.42	0.43	0.43	-0.46	718.64	720.28	-0.23	1361.00	1364.70	-0.27
	4	31.82	31.90	-0.26	0.43	0.43	-0.27	732.43	733.21	-0.11	2093.40	2097.90	-0.21
	3	31.82	31.81	0.04	0.43	0.43	-0.08	732.42	732.11	0.04	2825.80	2830.10	-0.15
	2	31.82	31.73	0.27	0.43	0.43	-0.07	732.40	731.54	0.12	3558.20	3561.60	-0.10
	1	31.82	31.73	0.28	0.43	0.43	0.01	762.35	761.43	0.12	4320.60	4323.00	-0.06
	B	31.82	31.76	0.17	0.43	0.43	0.50	694.92	693.70	0.18	5017.80	5016.70	0.02
Hollister	6	6.97	7.01	-0.51	0.24	0.24	-1.81	209.89	206.32	1.73	209.89	206.32	1.73
	5	6.97	7.00	-0.41	0.24	0.24	-1.34	234.37	229.17	2.27	444.26	435.49	2.01
	4	6.97	6.99	-0.22	0.24	0.24	-0.72	239.02	235.04	1.69	683.29	670.53	1.90
	3	6.97	6.97	0.06	0.24	0.24	0.28	239.02	241.89	-1.19	922.31	909.36	1.42
	2	6.97	6.96	0.25	0.24	0.23	1.17	238.95	251.88	-5.13	1161.30	1153.70	0.66
	1	6.97	6.95	0.26	0.24	0.23	1.05	248.79	262.41	-5.19	1410.00	1408.80	0.09
	B	6.97	6.95	0.27	0.24	0.24	0.13	226.72	227.80	-0.47	1629.30	1636.60	-0.45
Lexington	6	31.61	31.73	-0.39	0.58	0.59	-1.95	640.57	641.49	-0.14	640.57	641.49	-0.14
	5	31.60	31.72	-0.35	0.58	0.59	-1.55	717.71	720.26	-0.35	1357.50	1360.10	-0.19
	4	31.61	31.67	-0.20	0.58	0.58	-0.54	731.04	734.99	-0.54	2088.60	2093.40	-0.23
	3	31.61	31.57	0.10	0.58	0.58	0.16	731.12	733.64	-0.34	2819.70	2822.00	-0.08
	2	31.61	31.50	0.33	0.58	0.58	-0.02	731.44	732.71	-0.17	3551.10	3554.70	-0.10
	1	31.61	31.50	0.34	0.58	0.57	0.50	761.09	765.16	-0.53	4312.20	4319.80	-0.18
	B	31.61	31.53	0.24	0.58	0.58	-1.17	693.81	697.43	-0.52	4989.60	5005.90	-0.33
Lucerne	6	4.22	4.27	-1.29	0.74	0.74	1.05	160.69	162.96	-1.39	160.69	162.96	-1.39
	5	4.21	4.27	-1.20	0.74	0.74	0.52	179.79	183.51	-2.03	340.48	345.65	-1.50
	4	4.22	4.25	-0.93	0.74	0.74	0.18	183.24	187.51	-2.28	523.72	531.28	-1.42
	3	4.22	4.23	-0.43	0.74	0.75	-0.41	183.24	185.44	-1.19	706.96	711.62	-0.65
	2	4.21	4.22	-0.05	0.74	0.75	-0.72	183.23	187.35	-2.20	890.19	889.63	0.06
	1	4.22	4.22	-0.03	0.74	0.75	-0.72	190.71	194.76	-2.08	1080.90	1084.40	-0.32
	B	4.21	4.22	-0.17	0.74	0.74	0.97	173.85	173.64	0.12	1252.80	1257.90	-0.41

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Table 4: Peak response results of AMA-BI compared with NLTHA (continued)

Quake	Story	Displacement (cm)			Acceleration (g)			Force (kN)			Shear (kN)		
		AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)	AMA-BI	NLTHA	Error(%)
Newhall	6	17.14	17.23	-0.53	0.75	0.76	-0.18	386.02	385.64	0.10	386.02	385.64	0.10
	5	17.13	17.22	-0.48	0.75	0.75	0.14	431.91	429.35	0.60	817.93	814.99	0.36
	4	17.14	17.19	-0.31	0.75	0.75	0.25	440.19	437.16	0.69	1258.10	1252.10	0.48
	3	17.14	17.14	-0.04	0.75	0.75	0.06	440.18	440.75	-0.13	1698.30	1691.50	0.40
	2	17.14	17.11	0.17	0.75	0.76	-0.22	440.17	445.82	-1.27	2138.50	2135.40	0.15
	1	17.14	17.10	0.18	0.75	0.76	-0.28	458.17	464.09	-1.28	2596.60	2599.30	-0.10
	B	17.14	17.11	0.12	0.75	0.75	0.13	417.65	417.42	0.06	3013.80	3016.70	-0.10
Petrolia	6	31.02	31.17	-0.49	1.02	1.03	-0.57	629.31	639.89	-1.65	629.31	639.89	-1.65
	5	31.01	31.15	-0.46	1.02	1.03	-0.56	703.92	715.54	-1.62	1333.20	1355.40	-1.64
	4	31.01	31.10	-0.28	1.02	1.03	-0.36	717.48	724.81	-1.01	2050.70	2080.20	-1.42
	3	31.01	30.99	0.06	1.02	1.02	0.13	717.46	714.70	0.39	2768.20	2794.90	-0.96
	2	31.01	30.91	0.32	1.02	1.02	0.50	717.42	706.73	1.51	3485.60	3501.70	-0.46
	1	31.01	30.91	0.33	1.02	1.02	0.51	746.79	735.44	1.54	4232.40	4237.10	-0.11
	B	31.01	30.95	0.19	1.02	1.02	0.22	680.71	676.51	0.62	4908.60	4913.60	-0.10
Sylmar	6	30.87	30.99	-0.36	0.69	0.71	-2.42	625.74	624.90	0.13	625.74	624.90	0.13
	5	30.87	30.97	-0.32	0.69	0.70	-1.40	700.04	701.59	-0.22	1325.60	1325.70	-0.01
	4	30.87	30.92	-0.16	0.69	0.69	-0.47	713.42	715.94	-0.35	2039.10	2041.50	-0.12
	3	30.87	30.83	0.14	0.69	0.69	0.67	713.42	714.18	-0.11	2752.50	2755.60	-0.11
	2	30.87	30.75	0.37	0.69	0.68	1.49	713.43	711.65	0.25	3465.90	3467.20	-0.04
	1	30.87	30.75	0.39	0.69	0.68	1.19	742.57	739.75	0.38	4208.40	4206.90	0.04
	B	30.87	30.78	0.28	0.69	0.69	0.11	676.91	675.47	0.21	4889.00	4882.40	0.14
Yermo	6	7.94	8.01	-0.83	0.20	0.21	-2.22	225.47	227.85	-1.04	225.47	227.85	-1.04
	5	7.94	8.00	-0.78	0.20	0.21	-1.80	252.23	254.43	-0.86	477.70	482.23	-0.94
	4	7.94	7.99	-0.54	0.20	0.21	-0.90	257.08	258.30	-0.47	734.78	740.50	-0.77
	3	7.94	7.95	-0.04	0.20	0.20	0.28	257.08	256.63	0.18	991.86	997.13	-0.53
	2	7.94	7.91	0.35	0.20	0.20	0.69	257.07	255.47	0.63	1248.90	1252.60	-0.30
	1	7.94	7.91	0.37	0.20	0.20	1.17	267.59	265.99	0.60	1516.50	1518.60	-0.14
	B	7.94	7.93	0.14	0.20	0.20	1.80	243.91	243.39	0.21	1760.70	1762.00	-0.07

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