A NEW METHOD FOR NONLINEAR DYNAMIC ANALYSIS OF BASE ISOLATED STRUCTURES

Hamid MOHARRAMI¹, Navid NICKDOOST²

ABSTRACT

Base isolation has been one of the most effective approaches to structural control in the last few decades. Because of nonlinear behavior of base isolators, analysis of such structures requires efficient nonlinear analysis software that has both characteristics of speed and accuracy. Many classical approaches have been proposed for efficient nonlinear dynamic analyses, however this subject still needs more attention. In this paper a novel nonlinear dynamic analysis is proposed that not only increases the analysis speed significantly, but also gives more accurate results compared to those in the literature. The method is formulated based on release of extra internal forces that are mistakenly stored in linear analysis of nonlinear structures. To explain the method, one should remind that in linear analysis of structures, it is intuitively assumed that structural members have infinite capacities and the internal forces of members are proportional to the external loads.

Application of the proposed method to nonlinear analysis of several base isolated structures show that the proposed method results in much less computation time compared to classical methods. To illustrate the power and speed of the proposed method two examples have been included in the paper. It is shown that the proposed method can reach to a certain level of accuracy using larger time intervals compared to traditional classical methods.

Keywords: Base Isolation; Nonlinear Dynamic Analysis;

1. INTRODUCTION

Seismic control of structures is believed to be one of the most effective approaches in protecting structures against earthquake damages and mitigation of human disasters. As a well-known seismic control system, seismic base isolation has been used in several structures in many locations around the world. The isolation system reduces the interaction between two different surfaces. Isolating a structure is creating an interface that minimizes interaction between the ground and the structure (Connor 2003). Structural seismic isolation was perhaps conceived first by Touaillon in 1870, (US patent 99973). In 1909 Calantarients proposed a Talc layer in structure base level to provide sliding of base of structure on the ground (Bozorgnia et al. 2004). Two groups of isolators including rubber-based and sliding-base are generally addressed in the literature.

The philosophy of Seismic Isolation theory is simple. It is increasing the natural period of structure by implementation of a seismic isolator system with small lateral stiffness in the base level of structure. (Bozorgnia et al. 2004). Increasing the natural period of a structure reduces the effect of horizontal ground motions on the superstructure. The isolator in the base of a structure makes the superstructure not sense the ground motions and show somehow rigid behavior in the events of earthquake excitations. The almost rigid behavior will cause little internal forces in superstructure elements.

A base isolated structure is normally composed of a base on the ground, the base isolator system, a mat foundation and the superstructure. The behavior of base isolators is usually modeled as bilinear elastic-plastic. However, because of small transmitted lateral excitation, the superstructure behaves

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Several methods have been proposed for performing dynamic analysis of seismic isolated structures. One common method that is regularly used by scientists for dynamic analysis of base isolated structures is writing the superstructure and the base isolation system equations of motions separately. Employing some predefined models for base isolator elements, some hypothetical restoring forces are calculated for base isolators. These hypothetical forces that are calculated based on the pre and post yielding stiffness of the bearings and the displacements and/or velocities of the bearings will be applied to the structure base level and will be considered during dynamic analysis in an iterative manner.

Pondering the governing equations of biaxial interaction in hysteresis nonlinear behavior that was earlier investigated by Park (Park et al. 1986), and also considering mobilized forces for elastomeric bearings, Nagarajaiah proposed equations of motion for 3D isolated structure model (Nagarajaiah et al. 1991). He then employed Newmark dynamic method to solve the equations of motions. Matsagar et al. also investigated the response of base isolated structures during impact of adjacent structures (Matsagar et al. 2003). In another approach, Providakis considered an effective stiffness for the isolation system based on its bilinear behavior. (Providakis 2007)

Although there are some approximate approaches for dynamic analysis of base isolated structures, it is clear that the real and the most precise method in modeling and analysis of such structures is nonlinear dynamic analysis (NDA). The NDA methods are usually step by step algorithms (Clough et al. 2003).

All nonlinear dynamic analysis procedures are sequences of nonlinear static analyses in very small periods of time, with boundary conditions from the previous time step. In the nonlinear static analysis, due to deterioration of stiffness of some elements and reaching to the maximum capacity of some members, some unbalanced forces emerge. The nonlinear static analysis methods require the elimination of unbalanced forces. This is done by redistributing the unbalanced forces in the structure. Some nonlinear analysis methods that are based on Newton Raphson algorithm, also require the redefinition of stiffness matrix of the structure for redistributing the unbalanced forces. Sequential stiffness matrix redefinition and numerous iterations required for convergence in each time step, cause high calculation cost and analysis time and in some cases imprecise results.

In this paper a new method is proposed that not only its convergence is not conditional but also it is not iterative in each time step. It is formulated based on release of internal forces in excess of capacity of elements that have been falsely stored in structural elements. Moharrami and Riyazi developed this method for nonlinear static analysis of tension only and compression only members (Moharrami et al. 2000). Mahini et al. extended the method to static analysis of 2D frames (Mahini et al. 2013) and plane stress and plane strain problems (Moharrami et al. 2014). The use of this method, results in the more precise results of NDA without any need to redefine the stiffness matrix and unbalanced forces. It also requires much less calculation compared to other nonlinear analysis methods. In this paper, the method will be extended to nonlinear dynamic analysis of frames with one nonlinear degree of freedom at the base.

2. NONLINEAR DYNAMIC ANALYSIS

Because of little effect of earthquake on base-isolated superstructures it is wise to assume that the superstructure elements of isolated structure will not meet their elastic capacity limit and will remain elastic. But isolator elements show significant nonlinear behavior during earthquake excitation and accordingly they have to be modeled as nonlinear shear elements in NDA. Since the isolator elements are bounded between the base mat and the ground, and neither of them stretches due to earthquake, it is logical to consider that all the isolator elements behave in the same phase and there is only one nonlinear degree of freedom (DOF) at the structure base level. In this paper the isolator element has been assumed to have elastic–perfectly plastic behavior.
The proposed nonlinear dynamic analysis method consists of determining the amount of internal forces in nonlinear elements and releasing and redistributing the residual forces that, as a result of a linear dynamic analysis, are falsely stored in some elements. The true nonlinear analysis results will be calculated when the internal element forces that exceed the elastic capacities are released and redistributed. In other words, after applying horizontal forces of ground excitation and performing a linear analysis, the internal forces are computed. If these forces do not exceed the elastic capacity of elements, the element behavior will remain elastic; otherwise the calculated forces are not real and elements cannot tolerate them in elastic phase and will enter the plastic phase. In the proposed method of nonlinear dynamic analysis, the force in base isolator in excess of its elastic capacity will be identified, released and redistributed.

As a summary of the proposed methodology, for a 2D frame with one nonlinear element, in each time step, first the structure will be analyzed linearly and the shear force in the base isolator will be calculated. If the shear force exceeds the base-isolator’s linear capacity then an unknown virtual force \( Q \) will be applied to it to diminish the fake extra internal force in the base isolator and to bring it back to the exact value (elastic capacity level). In this process the impact of nonlinear behavior of shear element will be considered in other members' internal forces and deflections without any need to redefine the stiffness or mass matrix. Finally total displacement of the structure will be computed using simultaneous loading consisting of actual external force and virtual external force.

It is essential to mention that the inspection of element internal forces and the virtual external force \( Q \) have to be in element’s local coordinate. Therefore, in dynamic analysis of structure with one nonlinear member (base-isolator), only the degree of freedom that belongs to the nonlinear base-isolator will be considered in dominant nonlinear dynamic analysis.

### 2.1 Formulation of the Method

The equation of motion of multi degree of freedom system under arbitrary excitation is:

\[
[M]{\ddot{u}} + [C]{\dot{u}} + [K]{u} = \{F\} \tag{1}
\]

Where \( M, C, K \) and \( F \) stand for mass matrix, damping matrix, stiffness matrix and force vector respectively. Force vector can be dynamic force caused by an actuator or ground excitation. Employing modal decomposition one can write equation of motion for any single degree of freedom (mode shape) as follows:

\[
M_n\ddot{Z}_n + C_n\dot{Z}_n + K_nZ_n = \phi_n^T F
\tag{2}
\]

In this equation \( \phi_n^T \) represents transpose of mode shape. Also \( M_n, K_n \) and \( C_n \) are modal mass, modal stiffness and modal damping of mode \( n \) respectively. Solving this single degree of freedom equation of motion and considering superposition principle, total displacement of any degree of freedom will be calculated as follows:

\[
\{u\} = \sum_{i=1}^{n} \phi_i Z_i \tag{3}
\]

And finally the internal forces of each element will be calculated using the following formula:

\[
\{P_{elm}\} = [K_{elm}]{u_{elm}} \tag{4}
\]

where, \( K_{elm} \) is local stiffness of the member.

If the result of elastic analysis shows that the internal force of a member exceeds its elastic capacity limit then virtual external forces \( q \) will be applied to the member to release the extra force that is falsely stored in the member and bring the internal force to its capacity level.

If a unite virtual force with the intensity of \( q \) (that is \( Q \) in Global coordinates) is applied to the structure then the internal force of the member \( P \) under simultaneous forces of external dynamic force \( F \) and virtual unknown external force \( Q \) will be as follows:
\[ \bar{P} = R + fQ \]  

Where \( R \) represents internal reaction force due to external force \( F \) and \( fQ \) is the effect of \( Q \) in the member in question.

\[ P_{R_i} = R_i + f_i Q_i - Q_i = \bar{P}_i - Q_i \]  
\[ P_{R_{i-1}} = \bar{P}_{i-1} - Q_{i-1} \]  
\[ \Delta P_R = \Delta \bar{P} - \Delta Q = \bar{P}_i - \bar{P}_{i-1} - \Delta Q \]  

Considering modal superposition principle, \( \bar{P}_i \) will be calculated as:

\[ \bar{P}_i = K_{elm} \ast (B_{elm} \phi_1 Z_1) + K_{elm} \ast (B_{elm} \phi_2 Z_2) + \cdots \]  

In the previous equation \( K_{elm} \) is the local stiffness of member and \( \phi \) is the mode shape vector of the structure. \( B_{elm} \) is the Boolean matrix of the element in question that extracts the degrees of freedom related to the element from the global displacement vector of the structure. In the following formula, Equation 9 is rewritten for the first mode shape in \( i^{th} \) time step:

\[ \bar{P}_{i_{mode1}} = K_{elm} B_{elm} \phi_1 e^{-\xi_1 \omega_1 t_i} \left( \frac{A_{D_1}(t_i)}{M_{m_1} \omega^2_{D_1}} \sin(\omega_{D_1} t_i) - B_{D_1}(t_i) \cos(\omega_{D_1} t_i) \right) \]  

In Equation 10, the index 1, represent first mode shape and it has to be noted that nodal force of member is the sum of the effect of all modes respecting modal superposition principle. Therefore, to compute \( \bar{P}_i \) for each member Equation 10 should be recalculated for all modes and summed up. The \( A_{D_1} \) and \( A_{D_{i-1}} \) in the previous equation will be calculated under simultaneous loading \( \bar{F} = F + Q \) as below: (Paz M. 2003)
\[ A_{D_i} = A_{D_{i-1}} + \left( \bar{F}_{i-1} - t_{i-1} \frac{\Delta F}{\Delta t_i} \right) I_1 + \frac{\Delta F}{\Delta t_i} I_4 \]  
(11)

\[ B_{D_i} = B_{D_{i-1}} + \left( \bar{F}_{i-1} - t_{i-1} \frac{\Delta F}{\Delta t_i} \right) I_2 + \frac{\Delta F}{\Delta t_i} I_3 \]  
(12)

In these equations \( I_1 \), \( I_2 \), \( I_3 \) and \( I_4 \) are some integrals that will be calculated as follows: (Paz M. 2003)

\[ I_1 = \int_{t_{i-1}}^{t_i} e^{\xi \omega \tau} \sin \omega_D \tau \, d\tau = \left. \frac{e^{\xi \omega \tau}}{(\xi \omega)^2 + \omega_D^2} (\xi \omega \cos \omega_D \tau + \omega_D \sin \omega_D \tau) \right|_{t_{i-1}}^{t_i} \]  
(13)

\[ I_2 = \int_{t_{i-1}}^{t_i} e^{\xi \omega \tau} \cos \omega_D \tau \, d\tau = \left. \frac{e^{\xi \omega \tau}}{(\xi \omega)^2 + \omega_D^2} (\xi \omega \sin \omega_D \tau - \omega_D \cos \omega_D \tau) \right|_{t_{i-1}}^{t_i} \]  
(14)

\[ I_3 = \int_{t_{i-1}}^{t_i} \tau e^{\xi \omega \tau} \sin \omega_D \tau \, d\tau = \left. \left( \tau - \frac{\xi \omega}{(\xi \omega)^2 + \omega_D^2} \right) I_2' + \left( \frac{\omega_D}{(\xi \omega)^2 + \omega_D^2} \right) I_1' \right|_{t_{i-1}}^{t_i} \]  
(15)

\[ I_4 = \int_{t_{i-1}}^{t_i} \tau e^{\xi \omega \tau} \cos \omega_D \tau \, d\tau = \left. \left( \tau - \frac{\xi \omega}{(\xi \omega)^2 + \omega_D^2} \right) I_4' + \left( \frac{\omega_D}{(\xi \omega)^2 + \omega_D^2} \right) I_2' \right|_{t_{i-1}}^{t_i} \]  
(16)

In the previous equations \( I_1' \) and \( I_2' \) are Equations number 13 and 14 before entering boundary values. Substituting for \( A_{D_1} \) and \( B_{D_1} \), from Equations 11 and 12, Equation 10 can be rewritten as follows:

\[ \bar{P}_{t_{mode_1}} = K_{elem} B_{elem} \phi_r e^{-\xi \omega t_i} \left[ \left( A_{D_{i-1}} + \bar{F}_{i-1} I_{11} + \frac{\Delta F_{i1}}{\Delta t_i} (I_{41} - t_{i-1} I_{11}) \right) \sin(\omega_D t_i) - \left( B_{D_{i-1}} + \bar{F}_{i-1} I_{21} + \frac{\Delta F_{i1}}{\Delta t_i} (I_{31} - t_{i-1} I_{21}) \right) \cos(\omega_D t_i) + \frac{\phi_r^T}{\Delta t_i} \left[ \left( I_{41} - t_{i-1} I_{11} \right) \sin(\omega_D t_i) - \left( I_{31} - t_{i-1} I_{21} \right) \cos(\omega_D t_i) \right] \right] \]  
(17)

In Equation 17, \( \Delta q \) is the virtual unknown external force vector applied to the system in the global coordinates. All elements of this vector are zero except those corresponding to members that have reached to their nonlinear behavior. Also \( \bar{F}_{i-1} \) and \( \Delta F_i \) will be calculated as follows:

\[ (\Delta F_i)_r = (\phi_r)^T \{ F \}_{i-1} - (\phi_r)^T \{ F \}_{i} \]  
(18)

\[ (\bar{F}_{i-1})_r = (\phi_r)^T \{ F \}_{i-1} + (\phi_r)^T \{ Q \}_{i-1} \]  
(19)

In these equations \( r \) indices represents mode name and also \( \phi_r \), \( F \) and \( Q \) are \( \text{nadf} \times 1 \) vectors where \( \text{nadf} \) is the Number of Active Degrees of Freedom of the system and \( Q \) is the external virtual force vector applied to the structure in the global coordination that is computed as follows:

\[ \{ Q \}_i = \{ Q \}_{i-1} + \{ \Delta Q \}_i \]  
(20)

After calculating \( \bar{P}_i \) for each member, one can compute the variation of internal member force as follows:

\[ \{ \Delta P \}_r = \{ \Delta \bar{P} \} - \{ \Delta q \} = \{ \bar{P} \}_i - \{ \bar{P} \}_{i-1} - \{ \Delta q \} = \{ \bar{P} \}_i - \{ \alpha \} - \{ \Delta q \} \]  
(21)

In this equation \( \Delta q \) is the external virtual force applied to the member in its local coordination. In Equation 17, the following variables can be defined.

\[ e^{-\xi \omega t_i} \frac{M_m \omega_D^2 \omega_D^2}{M_m \omega_D^2} = N_j \]  
(22)
\[ \lambda_1 = \sum_{j=1}^{N_{\text{modes}}} K_{el} B_{el} \phi_j N_j \left( (A_{D_{i-1,j}} + F_{i-1,j}I_{l_1}) + \frac{\Delta P_j}{\Delta t} (I_{l_1} - t_{i-1}I_{l_1}) \right) \sin(\omega_{D_j} t_i) \]  

(23)

\[ \lambda_2 = \sum_{j=1}^{N_{\text{modes}}} K_{el} B_{el} \phi_j N_j \left( \frac{\phi_j}{\Delta t} (I_{l_1} - t_{i-1}I_{l_1}) \right) \sin(\omega_{D_j} t_i) \]  

(24)

\[ \lambda_3 = \sum_{j=1}^{N_{\text{modes}}} K_{el} B_{el} \phi_j N_j \left( (B_{D_{i-1,j}} + F_{i-1,j}I_{l_2}) + \frac{\Delta P_j}{\Delta t} (I_{l_2} - t_{i-1}I_{l_2}) \right) \cos(\omega_{D_j} t_i) \]  

(25)

\[ \lambda_4 = \sum_{j=1}^{N_{\text{modes}}} K_{el} B_{el} \phi_j N_j \left( \frac{\phi_j}{\Delta t} (I_{l_2} - t_{i-1}I_{l_2}) \right) \cos(\omega_{D_j} t_i) \]  

(26)

Using aforementioned variables and considering that each element has 6 degrees of freedom (two translational and one rotational at each end), Equation 21 can be rewritten as follows:

\[ \{\Delta P_R\} = \{\lambda_1\} - \{\lambda_3\} + [\lambda_2][\Delta Q] - [\lambda_4][\Delta Q] - \{\alpha\} - \{\Delta q\} \]  

(27)

In Equation 27, \(\lambda_1\), \(\lambda_3\), \(\Delta q\) and \(\Delta P_R\) are 6×1 vectors whereas \(\lambda_2\) and \(\lambda_4\) are 6×nadf matrices and \(\Delta Q\) is nadf×1 vector. It is important to note that all terms in the above equation have to be in a common coordinate system. In order to find the unknown variable of \(\Delta Q\) we can use the following condition for elastic perfectly plastic behavior of the isolator element.

\[ \{\Delta P_R\} + \{P_R\}_{i-1} \leq \{R_c\}_i \]  

(28)

\[ \{\Delta P_R\} + \{P_R\}_{i-1} \geq \{R_c\}_i \]  

(29)

In these equations \(\{R_c\}_i\) and \(\{R_c\}_i\) are member elastic capacity vectors in two alternative directions. Since, in this paper, only one member (base isolator) is assumed to have nonlinear behavior, therefore only the degree of freedom of the base isolator will be excited with virtual forces and thus corresponding base isolator element of \(\Delta Q\) vector will have nonzero value. By solving Equation 27 for this DOF the unknown variable will be obtained:

\[ [\lambda_2][B]([B][[\Delta Q]]) - ([\lambda_4][B])([B][[\Delta Q]]) - \{\alpha\} - \{\Delta q\} + \{P_R\}_{i-1} \leq \{R_c\}_i \]  

(30)

\[ [\lambda_2][B]([B][[\Delta Q]]) - ([\lambda_4][B])([B][[\Delta Q]]) - \{\alpha\} - \{\Delta q\} + \{P_R\}_{i-1} \geq \{R_c\}_i \]  

(31)

In these equations \(B\) is a Boolean matrix of the element in question that is employed to extract the related columns of \(\lambda_2\) and \(\lambda_4\) to the degrees of freedom of nonlinear element. Solving the previous equations for the unknown variables of global \(\Delta Q\) vector, will result two values for \(\Delta Q\) as below:

\[ [\Delta Q_1] \leq ([\lambda_2][B] - [\lambda_4][B] - [I])^{-1} * ([\{R_c\}_i + \{\alpha\} - \{\lambda_1\} + \{\lambda_3\} - \{P_R\}_{i-1}) \]  

(32)

\[ [\Delta Q_2] \geq ([\lambda_2][B] - [\lambda_4][B] - [I])^{-1} * ([\{R_c\}_i + \{\alpha\} - \{\lambda_1\} + \{\lambda_3\} - \{P_R\}_{i-1}) \]  

(33)

In the previous equations \([\Delta Q_1]\) and \([\Delta Q_2]\) are edf×1 vectors where edf is the degrees of freedom of the element with nonlinear behavior. According to the system conditions one of the obtained values for \(\Delta Q\) will be selected. This means that if the system is in positive plastic phase \(\Delta Q_1\) will be selected. Otherwise, if it is in negative plastic phase \(\Delta Q_2\) will be used. Also if the member is in elastic mode or \(\Delta Q_1 \cdot \Delta Q_2 < 0\) then \(\Delta Q\) value will be specified as zero.

After determining \(\Delta Q\) variable in local coordinate and also global coordinate then \(Q\) will be computed using Equation 20 and then \(\Delta P_R\) value will be calculated using Equation 27. Actual internal force of the member will be:

\[ \{P_R\}_i = \{P_R\}_{i-1} + \{\Delta P_R\}_i \]  

(34)
As mentioned before total displacement of the structure will be computed using simultaneous loading consisting of actual external forces and virtual external forces. So by calculating $F$ the total displacement of the structure will be computed as follows:

$$U_{ad} = \sum_{r=1}^{n} \sum_{i=1}^{n} \frac{e^{-\xi_r \omega_r t_i}}{M_r \omega_r} \left( A_{D_i r} (t_i) \sin(\omega_D r t_i) - B_{D_i r} (t_i) \cos(\omega_D r t_i) \right)$$

(35)

3. NUMERICAL EXAMPLE

To illustrate the capabilities of the proposed method in nonlinear dynamic analysis, some examples of base isolated structures are presented and compared to the results of commercial software SAP2000. For the first example, a two span two storey base isolated structure is modeled and analyzed under lateral ground excitation. Each span is assumed to be 5 meter and the height of each storey is 4 meter. Superstructure members are assumed to have standard section properties as mentioned in Table 1.

Table 1. First numerical example structural elements properties

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Unit</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>MPa</td>
<td>210000</td>
</tr>
<tr>
<td>$I$</td>
<td>Cm$^4$</td>
<td>5790</td>
</tr>
<tr>
<td>$A$</td>
<td>Cm$^2$</td>
<td>45.9</td>
</tr>
</tbody>
</table>

Uniform gravity loading of 500 kg/m is applied to the structure members. Base isolator behavior is assumed to be elastic perfectly plastic as shown in Figure 2.

![Figure 2. Bilinear elastic perfectly plastic behavior of isolator member](image)

The system is excited under north-south component of El Centro ground motion as shown in Figure 3.

![Figure 3. North south component of El Centro ground motion](image)

Figures 4 and 5 show the response of structure for the roof displacement and the isolator force.
resulting from application of proposed method and commercial software.

The interesting point about the proposed method is that it permanently eliminates any need to redefine the stiffness or mass matrix and thus it does not require sequential inverse of stiffness matrix. This is while the classic methods, such as Newmark method, require modification and inversing of the stiffness matrix in any iteration of the nonlinear phase. This results in higher analysis time and calculation cost compared to the proposed method.

4. COMPARISON OF THE PROPOSED METHOD WITH TRADITIONAL NONLINEAR ANALYSIS METHODS:

In order to further investigate and compare the proposed method with other classical nonlinear dynamic analysis methods another example as shown in Figure 6 is studied. (Paz M. 2003)
The single degree of freedom in Figure 6-(a) is subjected to the dynamic force as shown in Figure 6-(b) and has a nonlinear behavior as shown in Figure 6-(c). Other system properties are mentioned in Table 2.

Table 2. Second numerical example structural elements properties

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Unit</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>ksi</td>
<td>$30 \times 10^3$</td>
</tr>
<tr>
<td>$I$</td>
<td>in$^4$</td>
<td>100</td>
</tr>
<tr>
<td>$M$</td>
<td>k.sec$^2$/in</td>
<td>0.2</td>
</tr>
<tr>
<td>$\zeta$</td>
<td>-</td>
<td>0.087</td>
</tr>
</tbody>
</table>

To compare the precision of proposed method with one of classical nonlinear analysis the Newmark method is employed for nonlinear analysis of a single degree of freedom system, and its results is compared to the results of the proposed method. Figure 7 shows the responses of these two methods to the specified excitation.
Figure 7. Comparison of proposed method with classic nonlinear Newmark method

Figure 7 shows that the proposed method and the Newmark method result in almost the same dynamic response when the time step is very small (Δt = 0.01 second), but the results of bigger time steps (Δt = 0.1 and Δt = 0.2 seconds) are very different. In fact when Δt=0.1 seconds, the proposed method produces almost similar results to the result of Δt = 0.01 seconds. Whereas, when Δt=0.1 seconds, the results of Newmark method is far from its result of Δt = 0.01 seconds. It is concluded that the proposed method, unlike classical methods, is capable of finding more accurate responses even in bigger time steps. The ability of achieving appropriate results using bigger time steps compared to other conventional nonlinear analysis is outstanding aspect of the proposed method. It is interesting to note that if the number of engaged modes in the analysis is reduced according to the modal superposition principle, even more reduction in analysis time and cost will be possible.

5. SUMMARY AND CONCLUSION

Because of the extremely nonlinear behavior of base isolators during dynamic excitations, the most accurate approach to investigate the response of base isolated structures is to use nonlinear dynamic analysis methods. Time stepping methods are more common compared to other nonlinear dynamic analysis methods but the point is that the convergence of this methods depends on some special conditions. In order to obtain precise results, very small time steps should be used and as a result, the analysis and calculation time will be considerably increased. The following conclusions may be summarized from this article.

1- Since the proposed method does not require inversion of stiffness matrix even if the structure enters to nonlinear mode, the time of analysis is reduced considerably.
2- Figure 7 shows that the proposed method is able to find comparable results to conventional NDA methods using ten times bigger time steps.
3- The proposed method is unconditionally stable algorithm, because it really satisfies equilibrium in all time steps.

6. REFERENCES


