THE STRUCTURAL DESIGN OF TALL BUILDINGS
Struct. Design Tall Build. **10**, 321–334 (2001)
Published online in Wiley InterScience (www.interscience.wiley.com). DOI:10.1002/tal.179

SEISMIC ISOLATION DESIGN OF THE ARROWHEAD REGIONAL MEDICAL CENTER

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SUMMARY

The Arrowhead Regional Medical Center, located in Colton, CA, consists of six separate building structures containing a total of approximately 920 000 square foot of floor space for hospital and support services. Five of the six buildings are base-isolated. Each of the five base-isolated buildings is of different size and configuration, ranging from a six-storey, 360 000-square-foot curved-front nursing tower to a two-storey, 24 000-square-foot rectangular shaped central plant building. The building structures are framed with structural steel, utilizing concentric braced frames as the lateral force resisting system. The design ground motion for the site, which is located 3 km and 15 km from the San Jacinto and San Andreas faults, respectively, is very severe. A base-isolation system has been designed for this facility that will provide essentially elastic building response to the design strong ground motion. The base-isolation system is a hybrid passive energy dissipation system consisting of both linear and nonlinear and high damping rubber bearings along with viscous damping devices located at the base of the structure. The high damping rubber bearings provide both lateral stiffness, which governs the fundamental period of vibration of the system and hysteretic damping, while the viscous damping devices provide velocity-dependent damping, which serves to control overall building displacements. This will maximize the probability that this essential facility will remain fully operational after a major earthquake. Design criteria, structural analysis and design methodologies, and construction details are presented and discussed. The response of one of the base-isolated structures is calculated utilizing actual recorded ground motions from the 1994 Northridge Earthquake.

1. INTRODUCTION

The Arrowhead Regional Medical Center, located in Colton, CA, consists of six separate building structures containing a total of approximately 920 000 square foot of floor space for hospital and support services. Five of the six buildings are base-isolated. Each of the base-isolated buildings is of different size and configuration. The largest is a six-storey, 360 000-square-foot curved-front nursing tower. This is one of the four buildings which compose the main block of the hospital. The other three are a 274 000-square-foot diagnostic and treatment building, a 183 000-square-foot clinic building and the fixed-based MRI building. The project also consists of a 71 000-square-foot mental health centre and the central plant building. Figure 1 shows the site plan, with a brief description of each building. The building structures are framed with structural steel and concrete-filled steel deck. The lateral system of the superstructure consists of concentric braced frames as the lateral force resisting system. The design ground motion for the site, which is located 3 km and 15 km from the San Jacinto and San Andreas faults, respectively, is very severe.

The architects for the project are Bobrow/Thomas & Associates of Los Angeles in association with Perkins & Will of Chicago. KPFF Consulting Engineers in association with Taylor & Gaines,

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Received February 2001
Accepted February 2001

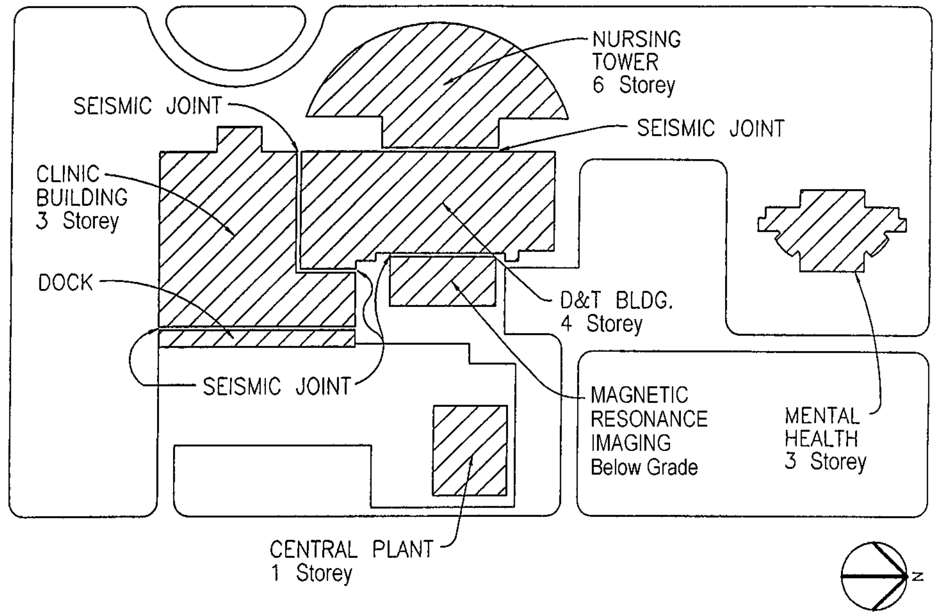


Figure 1. Site plan

Structural Engineers, are the structural engineers of record. The structural design of the project is governed by the California Code of Regulations, Title 24 (State of California, 1989).

A base-isolation system has been designed for this facility to provide essentially elastic building response to the design strong ground motion. The base-isolation system is a hybrid passive energy dissipation system consisting of both high damping and linear rubber bearings along with viscous damping devices (VDDs) located at the base of the structure. The high damping and linear rubber bearings provide the lateral stiffness, which governs the fundamental period of vibration of the system. Two forms of damping are incorporated into the system: hysteretic damping in the high damping rubber bearings and velocity-dependent damping in the viscous damping devices. The latter serve to control overall building displacements. Flexible mechanical joints are provided in the electrical and mechanical utility lines that lead into the buildings across the seismic gap. These joints have been tested to the maximum design displacement of 22 inches. This will help ensure that this essential facility will remain fully operational after a major earthquake.

2. DESIGN CRITERIA

2.1 Ground Motion

The design ground motion for this project consists of the following: a design-basis earthquake (DBE) response spectrum corresponding to a 10% probability of exceedence in a 50-year period, and a maximum capable earthquake (MCE) response spectrum that Title 24 (State of California, 1989) defines as the maximum event that can be postulated for the site. Based on a deterministic evaluation of the maximum capable earthquake response spectrum, utilizing the qualitative criteria noted above, the shape and magnitude of the MCE and DBE response spectra were determined to be equal. The MCE and DBE response spectra are shown in Figure 2. Three spectrum-compatible time-histories were generated that were selected as being representative of the geologic framework associated with

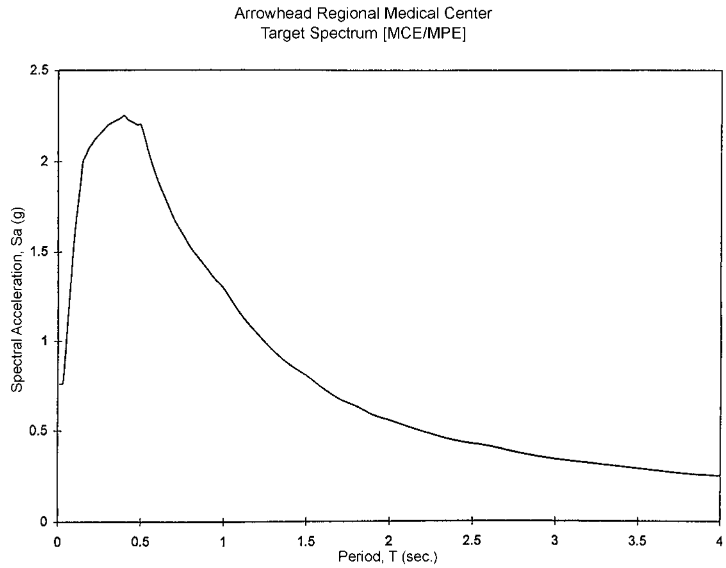


Figure 2. Maximum Capable Earthquake (MCE) target response spectrum

the Colton, CA, site. The three time-histories were: (1) 1979 Imperial Valley, Bonds Corner; (2) 1940 El Centro; and (3) 1952 Kern County. These time-history records, each with two orthogonal horizontal components, were scaled in the frequency domain to match the target response spectra and were then utilized as the input for the nonlinear time-history analyses. The maximum response resulting from the application of all three time-history records was then utilized for design, considering the worst-case load combination and point of application of inertial load.

2.2 *Structural Performance*

According to the California Office of Statewide Health Planning and Development (COSHPD) approved design criteria (KPFF, 1992), the superstructure (i.e. the structure above the lowest level diaphragms) was designed to remain 'nearly elastic' (as defined in (COSHPD, 1992) for the DBE. The superstructure was initially designed as a fixed-base structure (State of California, 1989). The maximum design base shear was the equivalent static base shear (0·21g) times a factor of 1·5 (0·31g) to account for 'near-fault' effects. This base shear was compared with the maximum base shear from the base-isolated nonlinear time-history analyses under the DBE.

2.3 *Maximum Displacement*

The maximum allowable displacement across the plane of isolation for the project is 22 inches. This maximum limit was imposed on the project for the following reasons: (1) 22 inches was approximately the upper bound for the technology of high damping rubber bearing displacement, which existed at the time of design; (2) 22 inches is approximately the upper bound for seismic joints, not only around the perimeter of the buildings but also between the buildings (i.e. 44 inches in any direction); and (3) 22 inches is approximately the maximum limit for the utility joints coming into the buildings.

2.4 Maximum Allowable Uplift

The maximum allowable uplift at the plane of isolation was jointly agreed upon by the design team and COSHPD. The criteria includes a limit on the maximum uplift displacement ($\frac{1}{2}$ inch),and the number of isolators that can uplift at any given instant of time. The seismic overturning forces and associated uplift displacements are accommodated by the braced-frame superstructure (Figure 3) and the 'spreader' truss which is located at the level directly above the base isolators (Figure 4).

3. STRUCTURAL ANALYSES AND DESIGN METHODOLOGIES

As stated previously, all the structural elements of the superstructure (e.g. braced frames, diaphragms, etc.) were designed and detailed as if they were part of a conventional fixed-base structure, with the following exceptions: (1) the braced-frame connections were designed for the maximum allowable compression load in the brace; (2) penalties imposed by Title 24 with respect to torsional irregularities and/or expected plastic behaviour in conventional fixed-base buildings were not enforced. The superstructures were analysed and designed using ETABS (Habibullah, 1989). The stiffness, mode shapes and mass matrices were extracted from the ETABS model for use in the base-isolation analysis as described in the next section.

4. ISOLATION SYSTEM ANALYSES

The high damping rubber isolators and viscous dampers have nonlinear inelastic properties, while the linear rubber isolators have linear elastic properties. Accordingly, a nonlinear dynamic model and computer program was needed for the analysis of the base-isolated buildings. A two-stage nonlinear model was employed for most of the analyses.

4.1 First model: structural model

The isolated building is represented by a three-dimensional, member-by-member, linear elastic structural model using the ETABS computer program (Habibullah, 1989). The model has a linearized representation of the isolation system to account for the stiffness of the isolators. This model is used to design the structure for base shear and overturning.

4.2 Second model: isolation model

This model is a reduced three-dimensional nonlinear model that accounts for the interaction between the elastic building and the spatially distributed nonlinear isolators that support the building (i.e. an elastic building model supported on a system of nonlinear base isolators). The building model is a reduced three-dimensional, linear model of the superstructure, where the building model is reduced to three degrees of freedom (DOFs) per floor level. The stiffness matrix or normal modes that define the dynamic characteristics of the reduced superstructure are obtained directly from the first model. Nonlinear models are used for the isolators and viscous dampers. The reduced three-dimensional elastic model of the superstructure is coupled to the isolators through the dynamic DOFs of the building model by compatibility matrices that relate isolator position and isolator deformation with respect to the building DOFs. Accordingly, the model incorporates the spatial distribution of the isolators, as well as their nonlinear, hysteretic characteristics. The 3D-BASIS computer program (Nagarajaiah Reinhorn and Constantinou 1991) was used for the analyses. However, to meet the specific needs of this project, extensive modifications were made to the 3D-BASIS computer program (KPFF, 1993).

Figure 5 shows a plan view of a typical isolator layout for the nursing tower, and an elevation view of a typical reduced three-dimensional model on base isolators. The high damping rubber isolators are

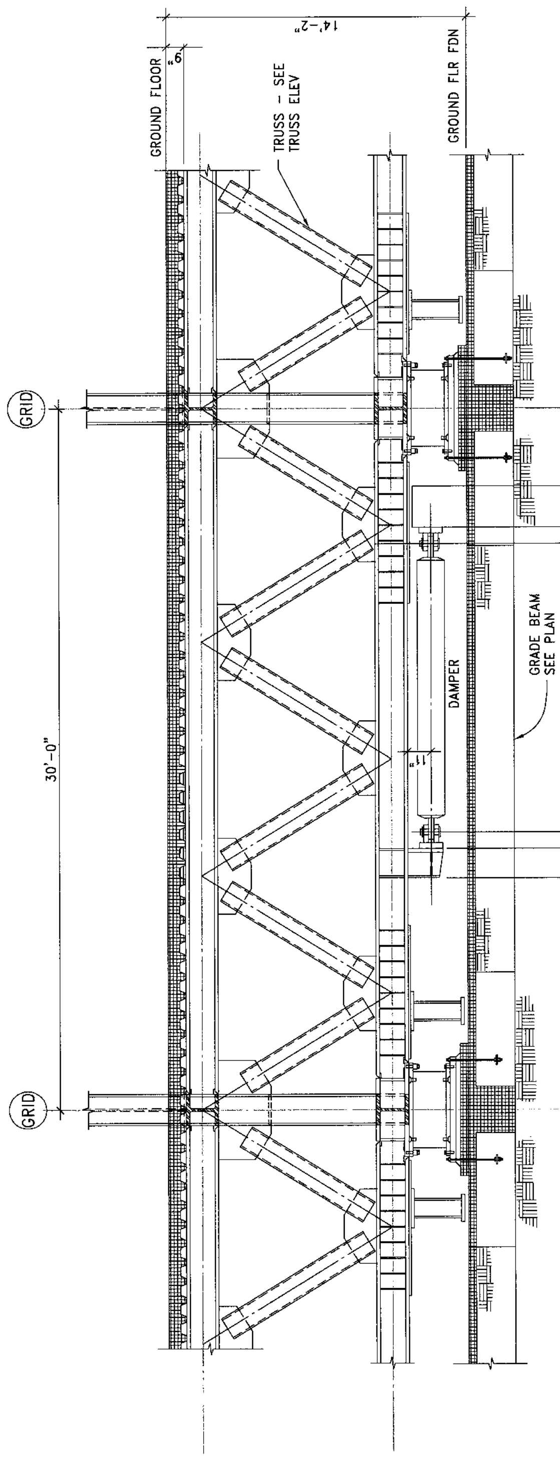


Figure 3. Typical braced frame

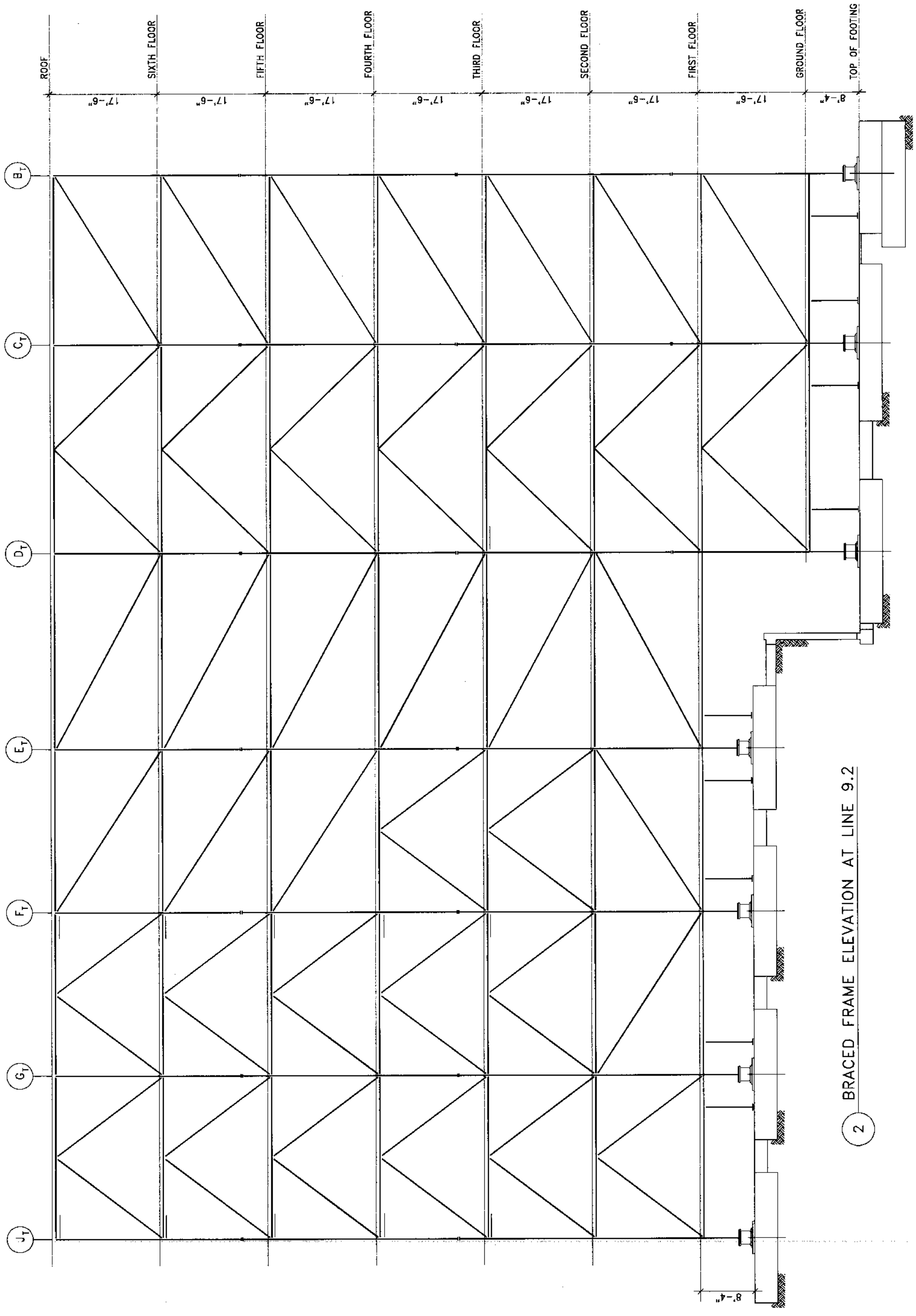
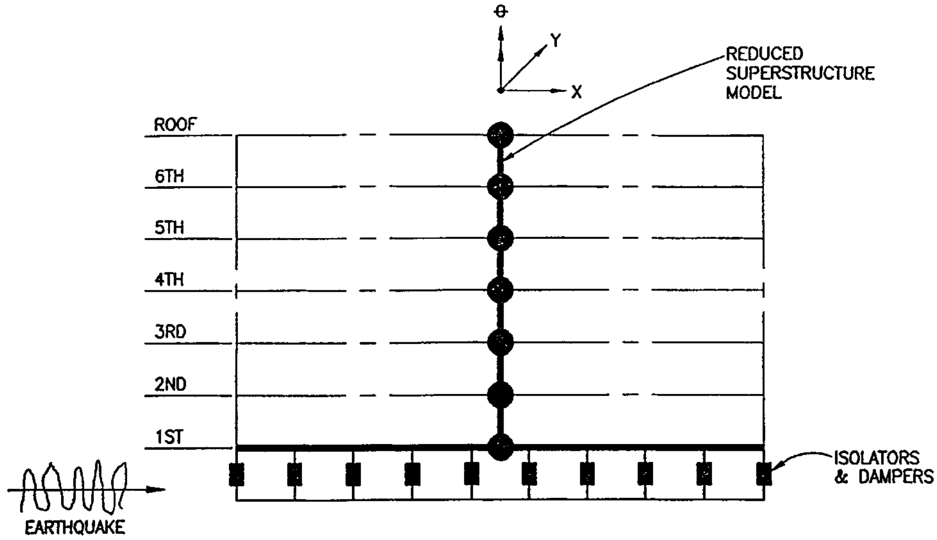
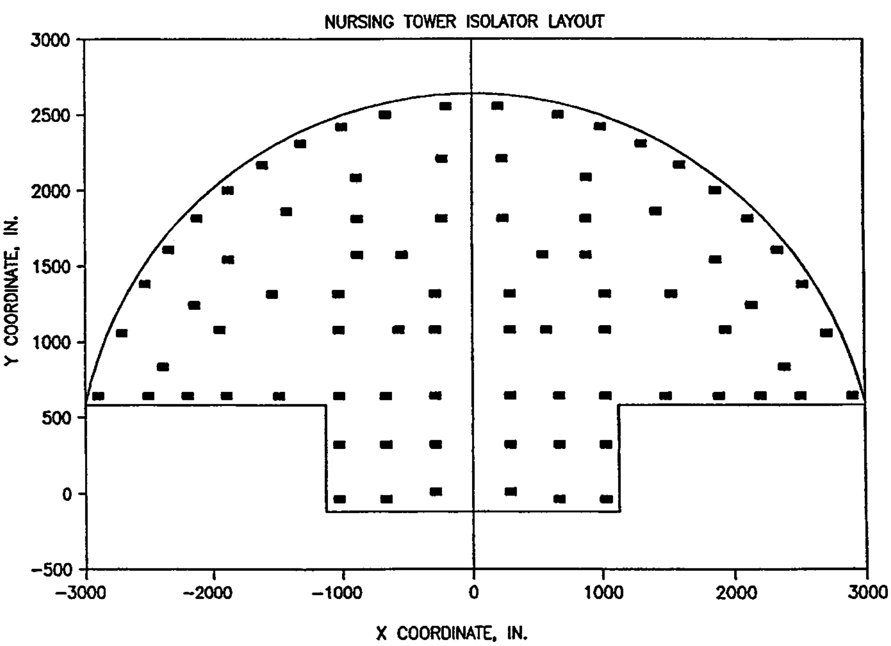


Figure 4. Spreader truss and viscous damping device (VDD) connection; ELEV, elevation; FDN, foundation



REDUCED THREE-DIMENSIONAL MODEL ON BASE ISOLATORS, ELEVATION VIEW.



PLAN VIEW OF A TYPICAL ISOLATOR LAYOUT, NURSING TOWER.

Figure 5. Reduced three-dimensional model on base isolators

modeled by a bilinear, hysteretic element whose properties are coupled in the two horizontal directions. The bilinear model is shown in Figure 6. The linear rubber isolators are modeled by a linear elastic element. The viscous dampers are modeled by a nonlinear element that represents the characteristics of the viscous damping devices which are intended for use on the project. The nonlinear

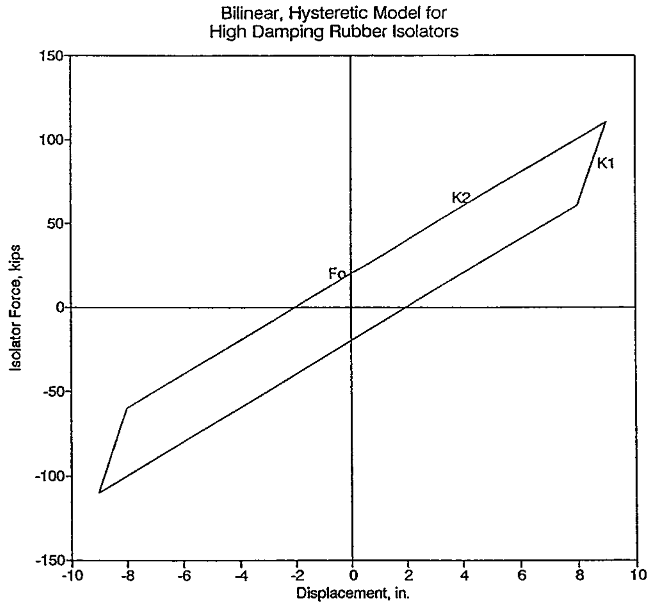


Figure 6. Bilinear, hysteretic isolator model. Fo, Force at Zero Displacement; K2, Yielded Slope; K1, Initial Slope

viscous damper element has different damping characteristics in two velocity ranges. In each velocity range the damping is defined by a damping coefficient, a velocity exponent and a constant damping force. The damping force provided by the element is shown in Figure 7, which shows typical upper and lower bound curves for viscous damping devices.

This model can be used to determine the deformation and force in the isolators, as well as the

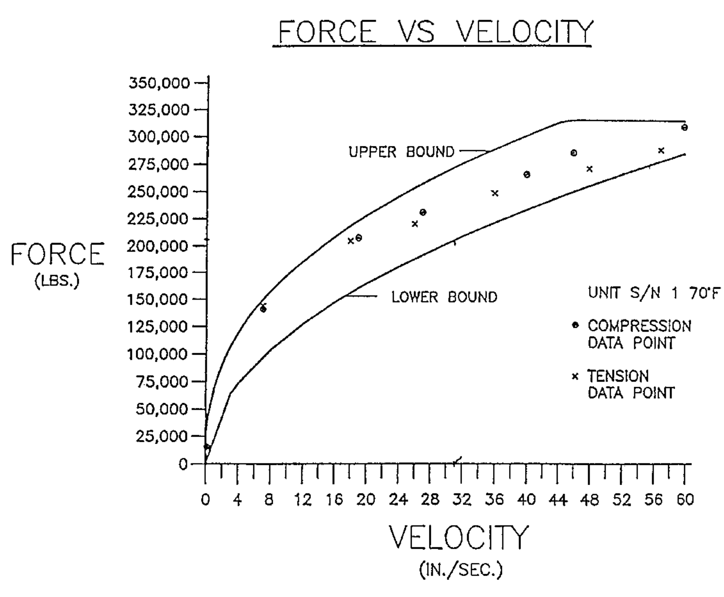


Figure 7. Nonlinear viscous damper model

interstorey drift and forces induced in the building model when it is subjected to two horizontal components of earthquake ground motions. Also, storey force distributions are determined at selected critical times, such as the times of occurrence of the maximum base shear and maximum overturning moments in the two horizontal directions, for use in later analyses.

4.3 *Combined model analysis*

The current reduced stiffness building model can not account for the vertical distribution of axial load on each isolator. Consequently, overturning was assessed using both models. Storey force distributions at selected critical times (obtained from the second model) are introduced into the three-dimensional building analysis model (first model) to obtain member forces and to assess overturning. If isolator uplift occurs, an iterative type of solution is employed in which the isolators that uplift are disconnected from the model and the analysis repeated until no additional uplift occurs. This is the area where use of a nonlinear three-dimensional model and computer program is clearly superior to the two-stage model.

Nonlinear dynamic analyses were conducted in support of the design process for all of the five base-isolated buildings. The analyses included five locations of the centres of mass, three sets of earthquake components for both the MCE and DBE, and two orientations of the earthquake components. In addition, analyses were undertaken to study the effects of parameter variations on system response and to minimize the effects of torsion. Many iterations of the building designs were required to arrive at the final structural and isolation system.

5. OVERTURNING ANALYSIS

The maximum overturning moments due to storey force profiles in the x and y directions are determined from the 3D-BASIS nonlinear analysis for each location of the displaced centre of mass (COM) (5% of the building plan dimension in each direction). The maximum base-level displacement is also determined for the above locations of the COM. Only the storey force profiles that correspond to the quadrants that produce the maximum displacements were run in ETABS to determine the maximum uplift. This is usually two opposite quadrants.

The uplift analysis procedure is performed by using ETABS. The static force profiles were applied in the appropriate quadrant and direction. All isolators were then checked for uplift by using an iterative procedure. Uplift was determined by checking for a positive axial force in the isolator or a positive deflection at its top. The properties of all isolators that uplift are modified to prevent them from taking axial load, while retaining their ability to transfer lateral loads. The ETABS model was then rerun. This procedure was repeated until the model became stable. During this iterative cycle, in addition to checking for isolators that uplift, a check is made for isolators that need to be reconnected. The amount of uplift was compared with the amount allowable as defined in the criteria. If the allowable amounts were exceeded, the uplift restraint system was stiffened and the overturning analysis was rerun.

6. CONSTRUCTION DETAILS

Because of the large axial loads and isolator displacements, special box columns needed to be detailed to handle the PΔ moments during the seismic event. Figure 8 shows these box columns in conjunction with the supplemental vertical load carrying system. Figures 4 and 9 show two variations of the VDD connection to the structure and the foundation. These details were designed for the maximum load viscous in the dampers of 315 kips.

7. ISOLATOR AND VISCOUS DAMPER TEST PLAN AND PROGRAM

Both the linear and high damping rubber bearings were tested under their maximum axial loads and

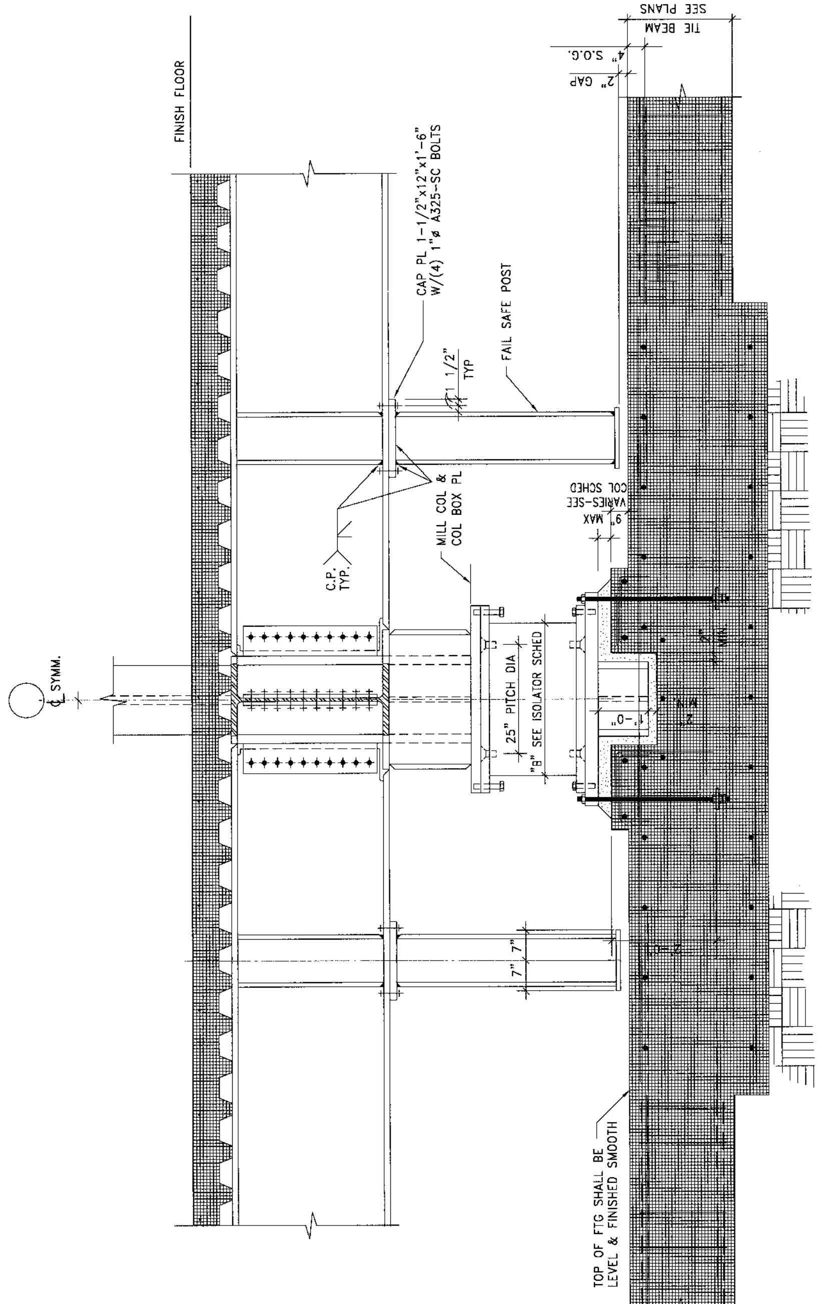


Figure 8. Box columns and isolator connection

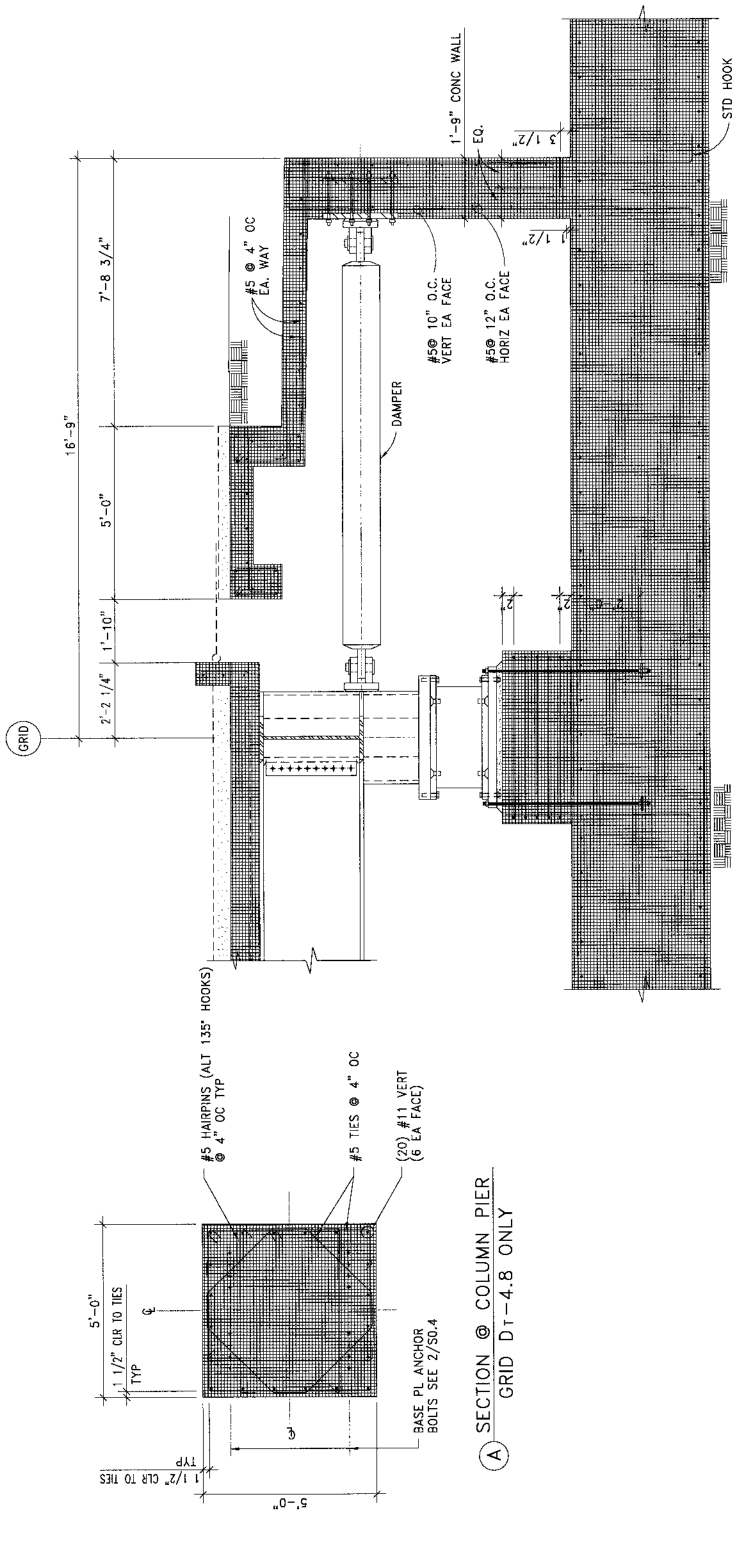


Figure 9. Viscous damping device (VDD) connection

shear displacement (20 or 22 inches) to verify their Lateral Force-Deflection properties and durability under the maximum seismic event. The bearings were axially loaded to (Dead Load + 0·5 × Live Load), (0·8 × Dead Load − 1·0 × Earthquake) and (1·2 Dead Load + 1·0 × Live Load + 1·0 × Earthquake) while being displaced for 10 cycles to 10, 15 and 20 inches. The VDDs had one-sixth-scale tests and full-scale tests. The one-sixth-scale tests were both cyclically loaded and drop tested to verify their properties. The drop tests and cyclic load tests were well correlated. The full-scale prototypes were only drop tested because the cyclic test would be cost prohibited and the one-sixth-scale tests showed they were not necessary. The drop tests were done at 10, 20, 30, 40, 50 and 60 inches per second. Figure 10 shows the test apparatus, and Figure 7 shows the test results.

8. ANALYTICAL RESPONSE TO SELECTED GROUND MOTIONS RECORDED FROM THE 1994 NORTHRIDGE EARTHQUAKE

The structural response of the six-storey nursing tower was quantified by the application of three actual time-history records obtained from the 1994 Northridge Earthquake. The time-history records which were utilized are: (1) Tarzana (CSMIP, 1994, Station 24436); (2) Sylmar (CSMIP, 1994, Station 24514); and (3) Arleta (CSMIP, 1994, Station 24087). These three time-history records were chosen based on their proximity to the epicentre as well as on the intensity and duration of the ground-shaking characterized by these recorded ground motions. In Figure 11, the response spectra associated with

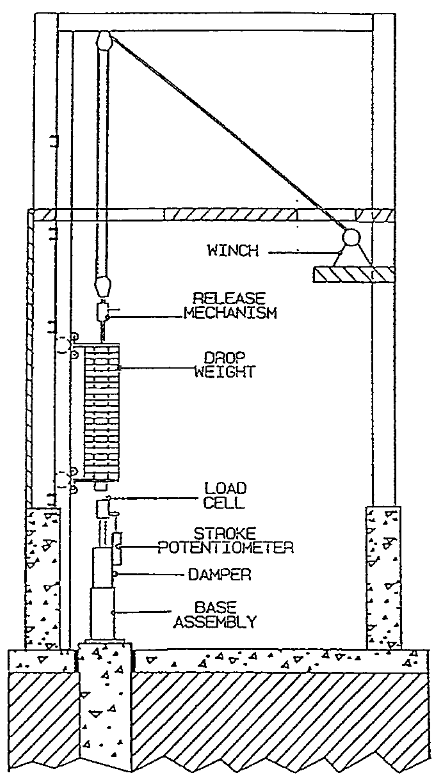


Figure 10. Viscous damping device (VDD) test apparatus

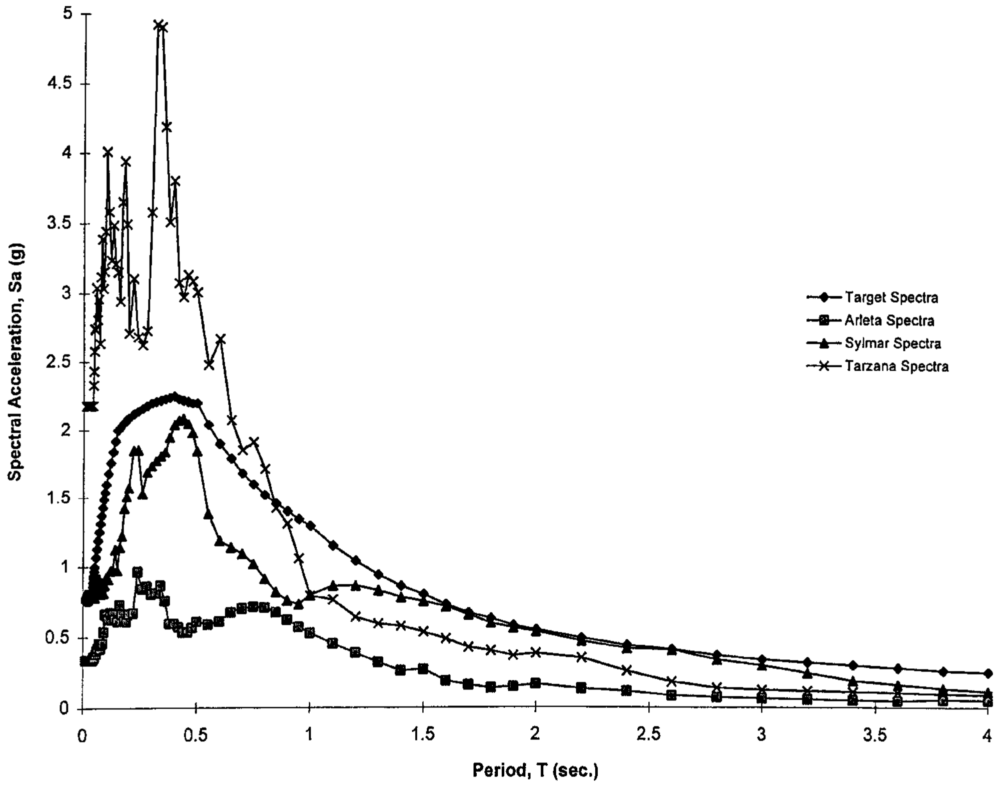


Figure 11. Comparison of Maximum Capable Earthquake target spectra to Northridge spectra

these time-history records are plotted against the maximum capable earthquake design spectra which was utilized for this project.

The unscaled and corrected horizontal time-history records were applied to the nursing tower analytical model at the calculated centre-of-mass location, and the actual tested properties of the elastomeric bearings and of the viscous damping devices were incorporated into the model. The results of these analyses are summarized in Table 1.

Table 1. Comparison of analytical response of the nursing tower for selected 1994 Northridge ground motions with design ground motions

	Structural response 1994 Northridge Earthquake Ground Motions			
Parameter	design ground motions	Tarzana	Sylmar	Arleta
Maximum centre of mass displacement (inches)	16·5	8·6	8·6	2·3
Maximum corner displacement (inches)	21·7	9·2	9·3	2·5
Maximum base shear (k)	14800 (0·31g)	13511 (0·29g)	12759 (0·27g)	7229 (0·15g)

9. CONCLUSIONS

The hybrid base-isolation system was essential in achieving the basic goal of finding an appropriate balance between building displacement and base shear under the severe site-specific ground motion. The high damping and linear rubber bearings provide the lateral stiffness which governs the effective period of the system. The hysteretic damping provided by the high damping rubber bearings and the velocity-dependent damping provided by the VDDs control the displacement. Without the VDDs, the rubber isolators would have to be stiffened to achieve the design displacement, resulting in an unacceptable base shear (i.e. $T \approx 1{\cdot}5$ s, and base shear $\approx 0{\cdot}8$g). By introducing the VDDs, the displacements were controlled to under 22 inches and the effective period was lengthened to about 3·0 s, thereby reducing the base shear to about 0·3g under the MCE. The extended period also significantly improves the response under smaller seismic events.

The calculated response of the nursing power due to the application of the three 1994 Northridge Earthquake horizontal time-history records indicates that the large velocity pulse which is incorporated in both the 1940 El Centro and the 1952 Kern County spectrum compatible time-history records governs the displacement response of the structure. This large velocity pulse was the primary motivation for the inclusion of the viscous damping devices into the base-isolation system. The viscous damping devices essentially control the displacement response of the structure to the desired level, while the acceleration enforce response of the system is governed by the forces-deflection properties of the elastomeric bearings. This velocity pulse is not nearly as strong in the recorded 1994 Northridge ground motions which were considered herein. Hence, the magnitude of the acceleration response which is obtained from the application of the 1994 Northridge time-history records approximates that which is obtained from the design ground motions, while the displacement response is significantly less relative to the design values. These results are encouraging in terms of ensuring that the future response of the various base-isolated buildings included in this project will be within the context of providing for continuity of operations after a major earthquake.

It is also interesting to note that the spectral accelerations associated with the Tarzana time-history record is 2–3 times that associated with the target response spectra for natural periods of less than 0·5 s. Hence, the anticipated response of the equivalent fixed-base structure would have been inelastic, with associated structural and nonstructural damage. This emphasizes the value of base isolation in ensuring essentially elastic response to a major earthquake.

REFERENCES

COSHPD (California Office of Statewide Health Planning and Development). 1992. An Acceptable Procedure for the Design and Review of California Hospital Buildings using Base Isolation. Building Safety Board, State of California, Sacramento, CA (revised edition of April 1989 publication).

CSMIP (California Strong Motion Instrumentation Program). 1994. Second Quick Report on CSMIP Strong-motion Data from the San Fernando Valley Earthquake of January 17, 1994. CSMIP, Sacramento, CA.

Habibullah A. 1989. ETABS, Three Dimensional Analysis of Building Systems. Computers and Structures Inc, February, Berkeley, CA.

KPFF. 1992. San Bernardino County Medical Center Replacement Project, Design and Testing Criteria for the Base Isolation System. November, KPFF Consulting Engineers, Santa Monica, CA.

KPFF. 1993. Modifications to Program 3D-BASIS. Report EA-9111-014, 17 August, KPFF Consulting Engineers, Santa Monica, CA (revised 21 September 1993).

Nagarajaiah S, Reinhorn M, Constantinou MC. 1991. 3D-BASIS Non-linear Dynamic Analysis of Three-dimensional Base Isolated Structures: Part II. Technical Report NCEER-91-0005, 28 February, National Center for Earthquake Engineering Research (NCEER), State University of New York at Buffalo, Buffalo, NY.

State of California. 1989. Title 24, Part 2, California Building Code of Regulations with Amendments. California Building Standards Commission, Sacramento, CA.

