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STUDY OF THE EARTHQUAKE RESPONSE OF THE BASE-ISOLATED LAW AND JUSTICE CENTER IN RANCHO CUCAMONGA

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SUMMARY

The recorded earthquake response of a base-isolated building—the Foothill Communities Law and Justice Center in Rancho Cucamonga—shaken by the 1985 Redlands earthquake (M_L 4·8) is discussed and analysed by employing system identification techniques. The calculated response of one-dimensional and three-dimensional linear structural models is fitted to the recorded motions of the superstructure using the '*modal minimization method*' for structural identification, in order to determine optimal estimates of the parameters of the dominant modes of the building. Simple one-dimensional analyses are used to identify also the effective values of key parameters (e.g. damping) of the isolation system. Furthermore, the recorded motions obtained from the densely instrumented foundation (i.e. below the isolation bearings) of the structure and from the free-field station located 330 ft‡ from the building show how the presence of the structure affects the incoming seismic waves. It is observed that the transverse component of motion (i.e. the component which is perpendicular to the long dimension of the plan of the building) is affected by the presence of the structure considerably more than the longitudinal component. Factors contributing to this effect are the extreme length of the structure (414 ft) and the rotational motions of the superstructure caused by the spatial variability of ground motion. It is pointed out that, despite the fact that the shift in the effective frequency of the structure induced by the isolation was very small, the elastomeric bearings were very effective in reducing the accelerations transmitted to the structure. This is attributed to the damping capacity of the isolation. Based on the observed response of the building to this small earthquake it can be stated with confidence that the structure performed according to expectations.

INTRODUCTION

Many of the problems associated with the response to strong earthquake shaking of traditionally designed structures—such as large inelastic deformations and associated severe and expensive structural and non-structural damage and inadequate protection of highly sensitive and costly equipment housed by modern structures—may be reduced considerably by restricting the earthquake-induced large deformations to special mechanical components. These components act as flexible pedestals that support the structure, and have a large capacity to absorb energy and act as a seismic fuse, thus preventing destructive horizontal motions from being transmitted into the structure and inhibiting resonances. This is the principle of base isolation (for a review on the subject see Kelly[1]).

The first building which was constructed in the U.S.A. employing base-isolation strategy for earthquake protection is the Foothill Communities Law and Justice Center in Rancho Cucamonga. The structure is located in the vicinity ($\sim$10 miles‡) of the junction of the San Andreas fault with the San Jacinto fault, where seventeen earthquakes of magnitudes 5·7 to 7·1 have occurred since 1890.[2] An earthquake of magnitude M_L 4·8 occurred on October 2, 1985 near Redlands, California, with the epicentre about 19 miles southeast from the site of the building. The earthquake did not cause any damage. It triggered, however, the sensors installed in the building by the Division of Mines and Geology (CDMG) of the State of California.[3] Five

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‡ 1 ft = 0·3048 m; 1 mile = 1·609 km.

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conventionally designed and constructed buildings in the area which were instrumented similarly amplified the ground acceleration from two to five times, as expected, while the base-isolated building reduced the ground acceleration by one-third, according to design.[4]

The purpose of the present paper is to study the recorded motions of this base-isolated building, employing techniques of structural identification.[5,6] The recorded earthquake response of the building is discussed and the dynamic characteristics of the superstructure and isolation system are identified and compared to the corresponding design values.

DESCRIPTION OF THE BUILDING, ISOLATION SYSTEM AND STRONG-MOTION INSTRUMENTATION

The Foothill Communities Law and Justice Center in Rancho Cucamonga, California, is a 4-storey building supported on elastomeric bearings which are interposed between the basement of the structure and the foundation (Figure 1). The structure above the isolation system (superstructure) consists of a steel space-frame stiffened at various bays by braced frames.[2] These braced frames resist lateral motion and provide enough rigidity so that the courthouse is expected to respond with a rigid body motion with estimated interstorey drifts of less than 1/2 in* under the worst seismic conditions (Maximum Credible Earthquake, Estimated Magnitude 8·3). Directly under the frames are 14 in concrete shear walls extending the full 14 ft height of the basement. These walls effectively form a 414 ft concrete box girder with the basement and first floor slabs as its webs, the longitudinal perimeter walls acting as flanges and the transverse shear walls as the web stiffeners of this girder. The purpose of this configuration is to spread the overturning reactions onto the bearings.[2] Furthermore, to keep the weight low, the exterior walls are glass-fibre-reinforced concrete panels.

The isolation system consist of eight different types of isolators that are placed under the building's 98 columns. These bearings have been designed to carry specific column loads that vary from 170 to 1,200 kips*

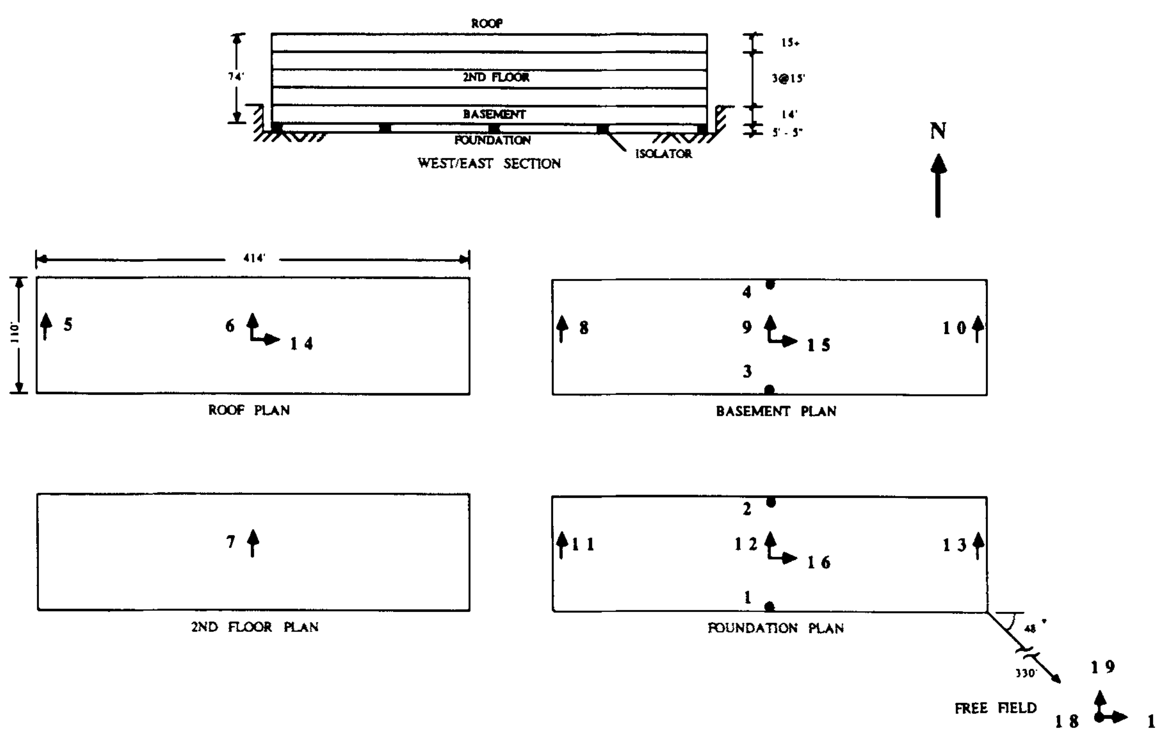

Figure 1. Sensor layout in the Rancho Cucamonga Law and Justice building. Arrows show the location and positive direction of the accelerometers. Dots indicate positive direction out of the plane of the figure (from Huang *et al.*[3])

* 1 in = 0·0254 m; 1 kip = 4·448 kN.

and can deform horizontally up to 15 in while maintaining the strength to support the vertical load. The isolators are 17 in high, 30 in in diameter and weigh 1,800 lb*. Two 1-1/2 in thick steel plates sandwich 23 layers of a special high damping rubber and 22 layers of 1/8 in steel plate.

The Law and Justice Center is instrumented by the California Strong Motion Instrumentation Program (CSMIP) with a total of 19 accelerometers, including a triaxial set of three sensors (Nos. 17, 18, 19, Figure 1) deployed about 330 ft from the building to record free-field ground motion unaffected by soil–structure interaction. The locations of the accelerometers are shown schematically in Figure 1. The sensor layout allows measurements of translational and rotational motions as well as overturning motions in the transverse (N–S) direction. The signals from each of the sensors are recorded on two multichannel recorders located in the centre of the building. Sensors 1–13 are connected to one recorder and sensors 14–19 are connected to a second recorder synchronized to begin recording simultaneously with the first within 0·1 sec, nominal.[3]

RECORDED EARTHQUAKE RESPONSE

The low-level accelerograms recorded at the courthouse during the October 2, 1985 Redlands, California, earthquake are near the minimum at which the system used to digitize the data is reliable. The data were filtered with a low-pass Ormsby filter with a corner frequency of 23 Hz (roll-off termination frequency of 25 Hz) and with a high-pass Ormsby filter with a corner frequency of 0·5 Hz (roll-off termination frequency of 0·4 Hz).[3] Noise analyses reported by Huang *et al.*[3] indicate that the results for these records are accurate for frequencies higher than about 1 Hz. Therefore, these data provide valuable information about the effectiveness of base isolation at low levels of high-frequency shaking. A preliminary analysis of the data has been presented by Huang *et al.*[7]

The longitudinal (E–W), transverse (N–S) and torsional acceleration and displacement response records along with the free-field records are shown in Figure 2. The torsional motions are accurate if the in-plane deformations of the floor slabs are negligible. The validity of this assumption has been verified for the basement floor.[8] Unfortunately, the rigidity of the slabs of the upper floors cannot be checked owing to the limited number of sensors. After a careful examination of Figure 2, the following observations can be made. Starting at the base of the building, it is evident that the high-frequency horizontal motions of the foundation (i.e below the isolators) were filtered out by the isolators and are not present in the corresponding motions of the basement (i.e. above the isolators). At the roof of the building, the translational motions recorded along the directions defined by the two principal axes (i.e. longitudinal and transverse) show characteristic differences. While the transverse (N–S) motion is almost monochromatic with frequency around 1·8 Hz during the entire duration of vibration, the longitudinal (E–W) motion has two distinct phases: the first 8 sec are dominated by a harmonic component with frequency around 3·5 Hz and the rest of the vibration exhibits the same frequency as the transverse motion (i.e. around 1·8 Hz). Finally, the torsional response of the building reaches its peak over the latter part of the excitation (see the rotational displacements of the roof in Figure 2). This probably can be attributed to surface waves which propagate with smaller velocities and arrive later on the record than the body waves.

Three factors contribute to the motions of base-isolated structures in general: (i) rigid body translation of the superstructure relative to the foundation, (ii) rocking and torsion of the building and (iii) structural deformation. For the base-isolated building under investigation, the horizontal deformation of the isolators is less than 0·05 cm along both the longitudinal and transverse direction, while the horizontal displacements of the roof relative to the basement are larger roughly by a factor of 4. Also, the rocking motions above as well as below the isolators were observed to be very small. Therefore, for this weak earthquake excitation, lateral structural deformation dominated over rigid body translation and rocking.[7]

Concluding, Figure 3(a) shows the rotational motions of the roof and basement while Figure 3(b) displays the torsional deformation of the superstructure. As Huang *et al.*[7] observe, the recorded data indicate that over the time interval 5 to 10 sec the rotational motions of the roof consist primarily of torsional deformations of the superstructure. However, over the time segment 14 to 22 sec more than half of the rotational response of the roof is due to rigid body rotation of the entire structure above the isolators.

* 1 lb = 0·454 kg.

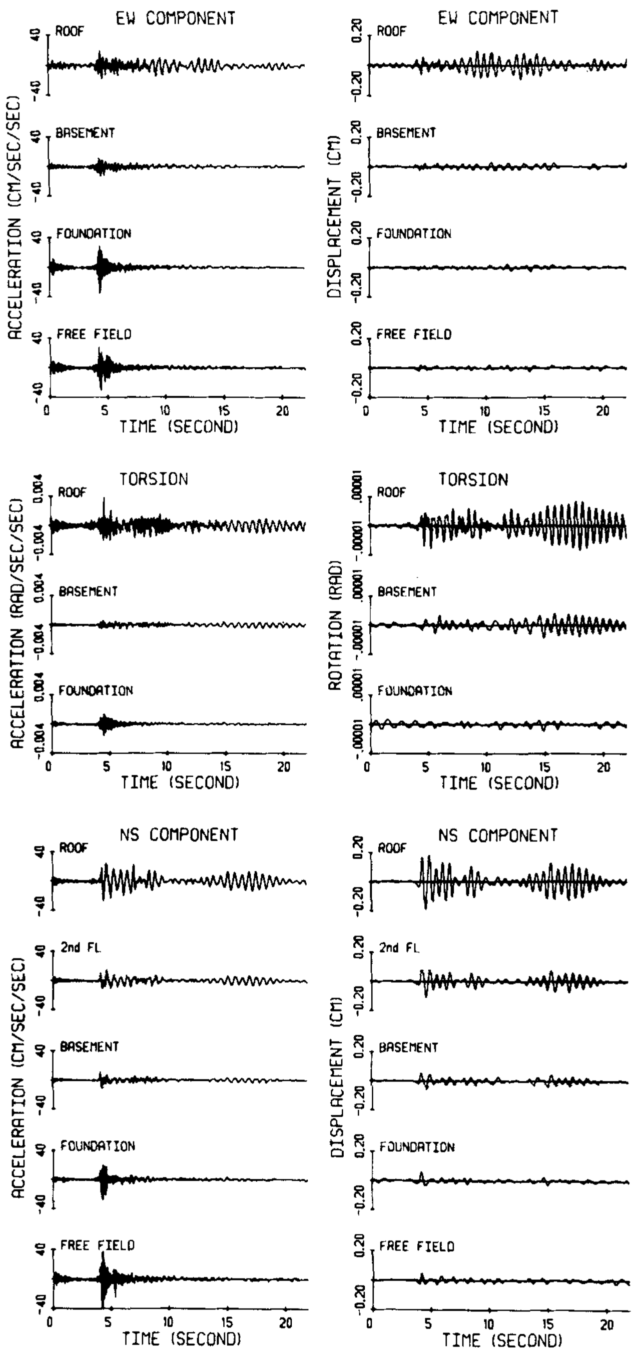

Figure 2. Acceleration and displacement profiles of the courthouse

1-D AND 3-D ANALYSES OF THE RECORDED RESPONSE

Estimates of the optimum values of the dynamic parameters (e.g. damping ratio, characteristic frequencies and corresponding modal shapes) of 1-D and 3-D linear mathematical models of the superstructure are obtained by applying the *modal minimization method* for structural identification. This technique, developed

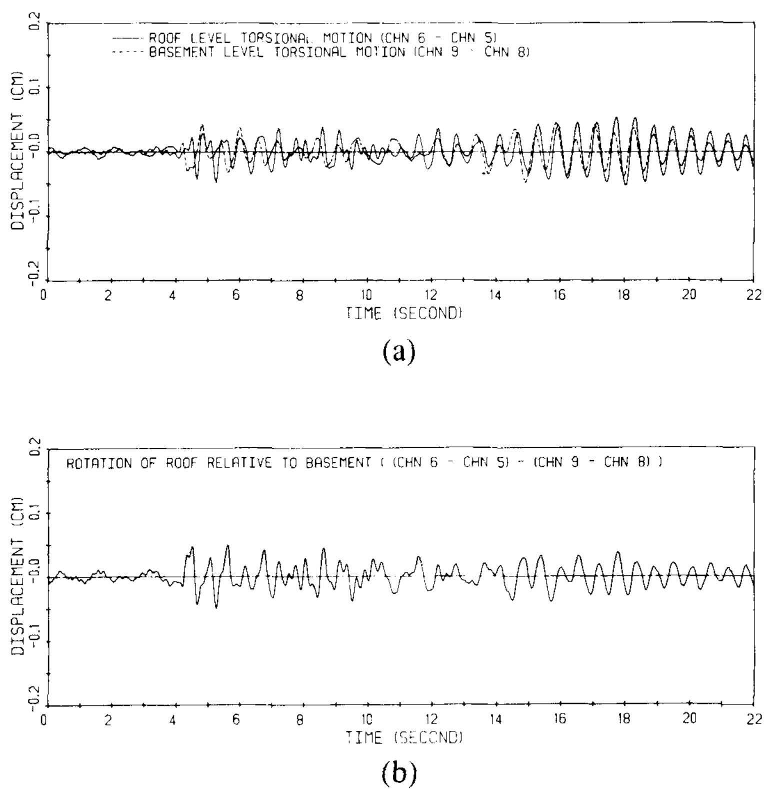

Figure 3. (a) The rotational displacements of the roof and basement and (b) torsional deformation of the superstructure. Over the time interval 5 to 10 sec the rotational motions of the roof consist primarily of torsional deformations of the superstructure, while over the time segment 14 to 22 sec more than half of the rotational response of the roof is due to rigid body rotation of the entire structure above the isolators

by Beck,[5] is a practical algorithm which minimizes some positive-definite measure-of-fit between the observed structural output and model output in the time domain, by systematically varying the model parameters.

We performed two types of analyses:

(i) One-dimensional (1-D) analyses considering a planar linear model with classical damping (see for example p. 556, Clough and Penzien[9]).

(ii) Three-dimensional (3-D) analyses considering an idealized model consisting of rigid floor decks supported on massless axially inextensible columns and walls, where for each floor are allowed three degrees of freedom, two orthogonal translations plus a rotation. The equations of motion of the mathematical model were formulated assuming that the centres of mass of all the floors lie on a vertical axis and the principal axes of resistance of all storeys are identically oriented. These were considered to be reasonable assumptions since the geometry of the floors of the building did not vary with height. However, no assumption was made related to the position of the centre for resistance of each floor.

The modal equations of the above two models are summarized in References (8, 10 and 11). Preliminary estimates of the modal frequencies of the building were obtained from the unsmoothed transfer functions. In the following we present the results of these analyses.

We considered for analysis the time segment 2–12 sec, covering the part of the response over which the roof displacements are predominantly due to structural deformation (Figure 3) and the signal-to-noise ratio of the excitation was judged to be high enough. Furthermore, as Beck[5] points out, the determination of effective participation factors is ill-conditioned for later portions of the records because the basement acceleration is small for these time intervals and the structural motion is dominated by the free-vibration component, which does not depend on the effective participation factor.

The vibration properties of the building for each of the two orthogonal directions—transverse (N–S) and longitudinal (E–W)—were estimated initially by fitting one-mode linear-time-invariant models. The estimated parameters obtained from these 1-D analyses are summarized in Table I. The quality of fitting was visually assessed to be satisfactory.[8] A one-mode model for each translational direction describes the observed response satisfactorily and the fundamental frequencies of the structure along the two principal directions appear to be identical, equal to 1·96 Hz. It is interesting to point out that the apparent frequency of 3·7 Hz of the longitudinal (E–W) component over the time segment 4–8·5 sec (Figure 2) is not a natural frequency of the structure, as was confirmed by checking also the corresponding transfer function. Simply, over the above time interval the superstructure is responding in the longitudinal (E–W) direction to an input acceleration [Channel 15 in Figure 4(c)] with a dominant harmonic component of frequency equal to 3·7 Hz.

Next, we proceeded to fit the response of the isolation system. In Figures 5(a) and (b), the simulated response of the basement is compared to the recorded motion. The quality of fitting achieved for the longitudinal (E–W) component is satisfactory. On the contrary, the simulation of the transverse (N–S) component is poor except for three pulses—at 4·25, 7 and 8 sec—for which the calculated response matches well with the recorded motion. The above observations, which are related to the elongated shape of the building and the spatial variability of ground motion, will be discussed in the next section. The inferred parameters of the isolation system are summarized in Table II. The value of the parameter f_b, which is estimated to be around 3·7 Hz, is the dominant frequency component of the basement motion [Figures 4(a) and (c)].

The damping factor of the elastomeric bearings ξ_b is estimated to be in the range 16–24 per cent. These values are fairly realistic for the following reasons: (i) the strains that the bearings were subjected to during the Redlands earthquake were very small (0·2 per cent), (ii) it was determined experimentally that for 2 per cent strains $\xi_b = 18$ per cent[2] and (iii) a characteristic of the elastomeric bearings used for the isolation of the courthouse is that the degree of damping increases with decreasing strain amplitudes.[2]

We concluded our investigation by performing a 3-D analysis of the response of the superstructure. The inferred parameters are summarized in Table III, which also shows the improvement in simulating the recorded response as higher modes are introduced progressively. In Figure 6 we compare the calculated response with the recorded motions. We used only the first three modes because the contributions of the fourth and fifth modes in improving the fitting are minimal, as can be judged by examining the progressive reduction of the normalized error. The quality of fitting for the two translational components is satisfactory.

Table I. Vibration characteristics of the superstructure of the Foothill Communities Law and Justice Center in Rancho Cucamonga, inferred from 1-D, one-mode model analyses, of the time interval 2–12 sec

	Period T_1 (sec)	Damping ξ_1 (%)	Effective participation factor $p^{(1)\dagger}_{\text{roof}}$	Normalized error J
EW	0·51 (1·96)*	6·1	1·672	0·125
NS	0·51 (1·96)*	4·4	1·417	0·335

*$f_1 = 1/T_1$.
†Defined as in Reference 6.

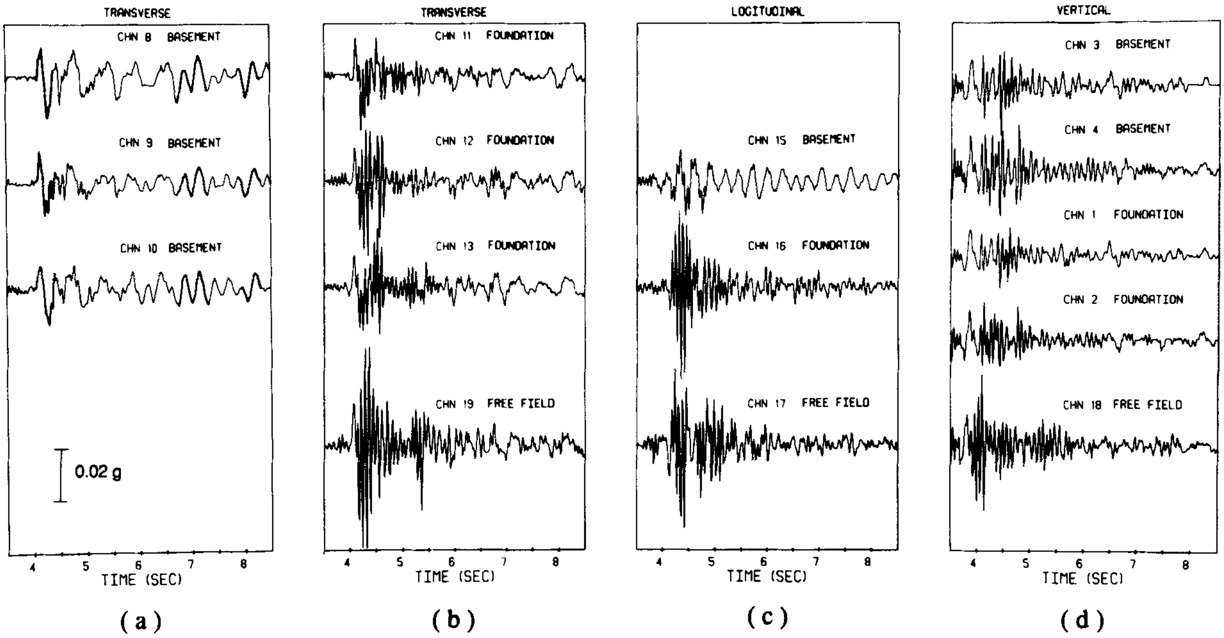

Figure 4. Accelerations recorded by the sensors which are located at the basement and foundation of the courthouse and in the free field

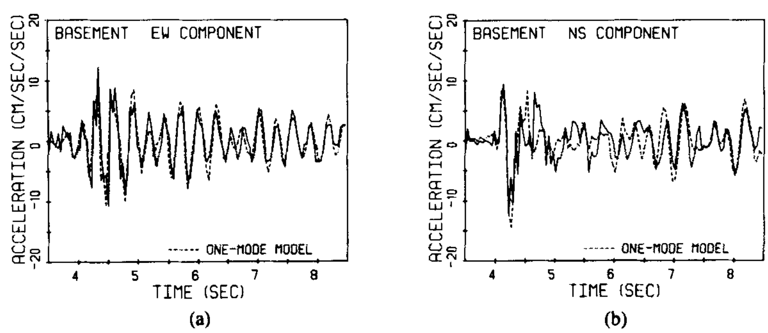

Figure 5. Comparison of the calculated absolute accelerations of the basement (———) with the recorded accelergrams (- - -) along both principal axes: (a) longitudinal and (b) transverse

However, simulation of the torsional response is considerably less successful, probably because the signal-to-noise ratio is small (estimated to be less than 2; see Figure 7 of Huang *et al.*[3]). We should point out that there is a considerable amount of transverse (N–S) motion in the first longitudinal (E–W) mode, and longitudinal (E–W) motion in the first transverse (N–S) mode (Table III). The proximity of these two modes in the frequency domain raises the possibility that each one of them actually is a linear combination of two modes much more one-dimensional in behaviour. We believe, however, that the inferred coupling reflects a true feature of the structure. To support this conjecture we refer to the ambient vibration measurements made on the courthouse by Pardoen and Hart[12] who observed that unsymmetrically placed K-braces introduced eccentricities and shifted the centre of rigidity east of the geometrical centre of the building by as much as 30 ft.

As a final note we compare the values of the modal parameters we inferred from the recorded earthquake response of the structure to the values inferred from ambient vibration measurements mentioned above[12] (Table IV). Noting that the ambient vibration measurements were conducted on essentially the structural

Table II. Vibration characteristics of the base-isolation system of the Foothill Communities Law and Justice Center in Rancho Cucamonga, inferred from analyses of the time interval 3·5–8·5 sec

	Period (sec) T_1 (sec)	Damping ξ_1 (%)	$c_v^\dagger$	$c_d^\dagger$	Normalized error J
EW	0·27 (3·70)*	15·7	−0·003	0·105	0·344
NS	0·27 (3·70)*	24·0	−0·005	1·195	0·716

*$f_b = 1/T_b$.
†Assuming vibration of the superstructure in the first mode, the equation of motion of the basement floor may be written as: $(\ddot{u}_b + \ddot{u}_g) + 2\xi_b\omega_b\dot{u}_b + \omega_b^2 u_b = c_V\dot{u}_R + c_d u_R$, where u_R and $\dot{u}_R$ are displacement and velocity, respectively, of the roof relative to the basement.

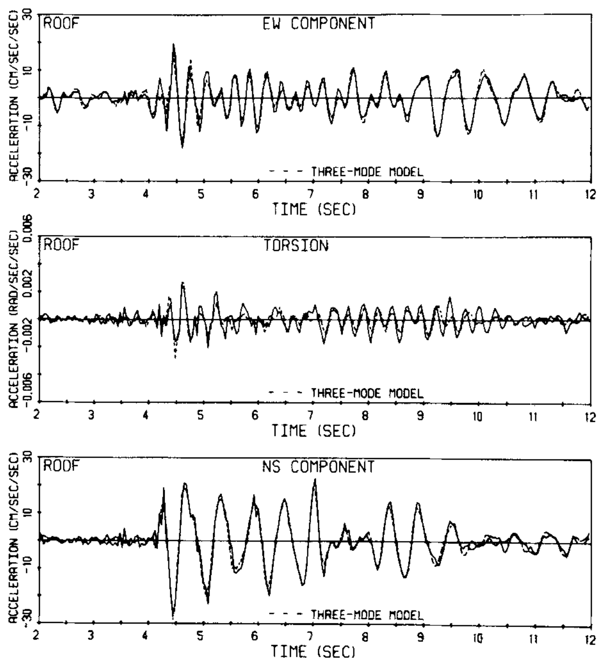

Figure 6. Accelerations recorded (———) at the roof of the courthouse and calculated absolute accelerations (– – –) of the optimal 3-D model determined by matching simultaneously all three components (transverse, rotational and longitudinal) of acceleration

skeleton of the building (i.e. on the structure without its skin), the differences in the inferred values of the characteristic frequencies are attributed to the stiffness of the skin of the superstructure. While the inferred damping values of the two translational modes are in good agreement (suggesting that the skin of the structure responded perfectly elastically to the weak earthquake motions), the damping value of the torsional mode we inferred from the earthquake data is low relative to the value from ambient vibration measurements 0·7 vs. 2·1 per cent). This rather large difference may be attributed, at least partially, to the low signal-to-noise ratio (especially for the torsional motions) noted above for the earthquake data, although errors associated with the half-power point method used in the ambient vibration measurements cannot be excluded.

DISCUSSION

In the previous section we attributed the limited success in simulating successfully the transverse response of the base-isolation system [Figure 5(b)] to the elongated shape of the building and to the spatial variability of the ground motion. In order to investigate this point further let us take a closer look at the motions recorded at the free field (channels 17, 18 and 19), at the foundation level (channels 1, 2, 11, 12, 13 and 16) and the basement level (channels 3, 4, 8, 9, 10 and 15). These recorded motions are shown in Figures 4(a), (b), (c) and (d), where all were plotted using the same scale. We start by comparing the motions of the foundation with the free-field motions. Clearly, the amplitudes of the high-frequency waves of the transverse (N–S) component of the motions recorded at the foundation (CHN 11, 12 and 13) suffered a considerable reduction, which appears to be more pronounced for CHN 11 and 13 which are located at the edges of the long dimension of the foundation plan. On the contrary, the longitudinal (E–W) component of motion appears to be the least affected of all components of motion (compare CHN 16 with CHN 17), including the vertical component which also suffered a reduction (compare CHN 1 and 2 with CHN 18). It is evident from these data that an elongated structure like the courthouse affects the transverse component of ground motion much more severely than the longitudinal one. We attribute this effect to the rotational motion of the superstructure which is transmitted from the stiff basement, through the bearings, to the ground. We consider eccentricities and spatial variablity of ground motion to be the most important causes contributing to the rotational motions of the superstructure. The spatial variability of ground motion is caused either by non-vertically incident coherent seismic waves or by scattered (and incoherent) seismic energy. From simple correlation analysis of the direct P and S waves we found that the incoming waves at the site of the courthouse impinge on the free surface almost vertically. We conclude therefore that the spatial variation of ground motion in this case is primarily due to scattering. In fact, this conclusion appears to be very reasonable in view of the fact that the high-frequency (~ 10 Hz) components which dominate the free-field and foundation motions are known to be strongly scattered by crustal heterogeneities.[13,14]

Turning now our attention to the recorded motions of the basement we observe that in the longitudinal direction (CHN 15), the elastomeric bearings acted as a narrow band filter with centre frequency approximately equal to 3·7 Hz. In view of the high stiffness of the basement floor—both for shear and axial deformations—in the longitudinal direction, it is safe to assume that the record of sensor 15 is representative of the longitudinal component of motion of any point of the basement floor. On the contrary, the transverse component of motion varies along the length of the basement floor (CHN 8, 9 and 10). This variation is attributed to the spatially variable input motion at the foundation level and the torsional response of the superstructure. In Figure 4(a), we highlighted the pulses which, by visual inspection of CHN 8, 9 and 10, appeared to be the least distorted by scattering effects. These pulses, located at 4·25, 7 and 8 sec, are the only segments of the transverse motion of the isolation system which we modelled satisfactorily [see Figure 5(b)].

Finally, by comparing CHN 1 and 2 with CHN 3 and 4 we observe that the vertical motions were transmitted from the foundation to the basement relatively undistorted and slightly amplified. Despite this amplification, the vertical motions of the basement remained smaller in amplitude than that of the free-field motions.

Concluding this discussion, we would like to compare the inferred dynamic characteristics of the courthouse with the corresponding design values. The building has been designed to withstand an estimated 'Maximum Credible Earthquake' of magnitude $M8{\cdot}3$ that can occur along the San Andreas fault, 22 km from the building site. The design spectrum specified for such an earthquake has constant spectral accelerations over the frequency range 1·25–5·88 Hz. According to design estimates, for such a strong earthquake, the isolation system is expected to shift the frequency of the structure to a value around 0·5 Hz (=effective fundamental frequency of the entire system of superstructure plus base-isolation), which is out of the severe range of the spectrum of this earthquake.[2] However, the recorded response of the building to the 1985 Redlands earthquake shows that the effective frequency of the base isolated structure is around 1·7 Hz (this

Table III. Vibration characteristics of the superstructure of the Foothill Communities Law and Justice Center in Rancho Cucamonga, inferred from 3-D analyses of the time interval 2–12 sec

Model	Mode r	Period T_r (sec)	Damping ξ_r (%)	Participation factors $C^{(r)}_{\text{roof, EW}}$	Participation factors $C^{(r)}_{\text{roof},\theta}$	Participation factors $C^{(r)}_{\text{roof, NS}}$	Modal shapes $R^{(r)}_{\text{roof, EW}}$	Modal shapes $R^{(r)}_{\text{roof}\,\theta}$	Modal shapes $R^{(r)}_{\text{roof, NS}}$	Normalised error J
1 mode	1st EW	0·52 [1·92]*	0·9	1·490	0·038	−0·315	1·0	−0·20	−0·47	1·940
2 modes	1st EW	0·51 [1·96]*	2·6	1·533	0·015	−0·389	1·0	−0·21	−0·44	1·152
	1st NS	0·49 [2·04]*	2·9	0·373	−0·928	1·233	0·32	−0·05	1·0	
3 modes	1st EW	0·51 [1·96]*	2·5	1·534	0·098	−0·451	1·0	−0·23	−0·41	0·712
	1st NS	0·49 [2·04]*	2·8	0·339	−0·890	1·250	0·38	0·07	1·0	
	2nd Torsional	0·23 [4·35]*	4·3	0·170	0·304	0·481	−0·03	1·0	0·21	
4 modes	1st EW	0·51 [1·96]*	2·6	1·378	0·102	−0·467	1·0	−0·19	−0·41	0·596
	1st NS	0·49 [2·04]*	2·8	0·230	−0·772	1·214	0·40	0·12	1·0	
	2nd Torsional	0·23 [4·35]*	2·8	0·029	0·303	0·497	0·04	1·0	0·22	
	1st Torsional	0·43 [2·33]*	0·7	−0·100	0·164	−0·050	−0·43	1·0	−0·96	
5 modes	1st EW	0·51 [0·96]*	2·6	1·387	0·097	−0·476	1·0	−0·18	−0·38	0·521
	1st NS	0·49 [2·04]*	2·9	0·196	−0·695	1·212	0·40	0·14	1·0	
	2nd Torsional	0·23 [4·35]*	3·0	0·024	0·293	0·478	0·03	1·0	0·23	
	1st Torsional	0·43 [2·33]*	1·3	0·111	−0·228	0·018	0·82	−0·79	1·0	
	2nd EW	0·20 [5·00]*	4·3	−0·304	−0·066	0·051	1·0	−0·16	0·04	

*$f_r = 1/T_r$.

Table IV. Vibration characteristics of the superstructure of the Foothill Communities Law and Justice Center in Rancho Cucamonga, inferred from ambient vibration measurements[12]

Mode	Frequency (Hz)	Damping (%)
1st EW	1·64	2·4
1st NS	1·76	2·3
1st torsional	2·03	2·1

can be verified by the Fourier spectra of the roof motions). This value of the effective frequency can be obtained also from the inferred values of the fundamental frequency of the superstructure (~ 2 Hz) and the frequency of the isolation system ($\sim 3{\cdot}7$ Hz) by using Dunkerley's equation (see for example, page 276, Thomson[15]). The above discrepancy between the design value (0·5 Hz) and the observed value (1·7 Hz) of the effective frequency of the base-isolated structure is explained as follows. The design value of 0·5 Hz was based on the assumption that the rubber bearings will be stretched to high strains (~ 50 per cent)—as is naturally expected for a strong earthquake—at which the shear modulus of the high damping rubber takes values of about 100 psi.[2]* During the Redlands earthquake the strains were very small ($\sim 0{\cdot}2$ per cent). For such values of strain the shear modulus of rubber is about 1000 psi.[2] Thus the effective frequency of the base-isolated structure during this small earthquake should be equal approximately to $0{\cdot}5 \times \sqrt{1000/100} = 1{\cdot}6$ Hz, which is close to the observed value of 1·7 Hz.

CONCLUSION

Despite the small shift in the effective frequency of the Law and Justice Center in Rancho Cucamonga during the 1985 Redlands earthquake (M_L 4·8), the elastomeric bearings of its base-isolation system were very effective in reducing the accelerations transmitted to the superstructure. This is attributed to the damping capacity of the isolation. Therefore it may be concluded that the building performed according to expectations.

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* 1 psi = 6·895 kN/m^2.

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