Earthquake Damage Detection in the Imperial County Services Building III:

Analysis of Wave Travel Times via Impulse Response Functions

Maria I. Todorovska\textsuperscript{1} and Mihailo D. Trifunac\textsuperscript{2}

ABSTRACT The majority of structural health monitoring methods are based on detecting changes in the modal properties, which are \textit{global} characteristics of the structure, and are not sensitive to local damage. Wave travel times between selected sections of a structure, on the other hand, are local characteristics, and the methods based on detecting changes in such characteristics of response are potentially more sensitive to local damage. In this paper, one such method is explored using strong motion data from the 1979 Imperial Valley Earthquake recorded in the former Imperial County Services (ICS) Building, severely damaged by this earthquake. Shear wave travel times are measured from \textit{impulse response functions} computed for three time windows of the recorded seismic response – before, during, and after the largest amplitude response, as determined from previous studies of this building, based on analysis of novelties in the recorded response. The results suggest initial spatial distribution of stiffness consistent with the design characteristics, and reduction of stiffness following the major damage consistent with the spatial distribution of the observed damage. The travel times were also used to estimate the fundamental fixed-base frequency of the structure, $f_1$, and its changes during this earthquake. These estimates are consistent with previous estimates of the soil-structure system frequency, $f_{sys}$, during the earthquakes (always $f_1 < f_{sys}$), and with other estimates of frequency ($f_1$ from ETABS models, and $f_{sys}$ from ambient vibration tests, and “instantaneous” $f_1$ from high frequency pulse propagation).

\textbf{Keywords}: Damage detection; Structural health monitoring; Wave travel times; Deconvolution; Impulse response function; Earthquake response; Wave propagation in structures; Imperial County Services Building.

\textsuperscript{1} Research Professor, Dept. of Civil Engineering, University of Southern California, Los Angeles, CA 90089-2531. E-mail: mttodorov@usc.edu.

\textsuperscript{2} Professor, Dept. of Civil Engineering, University of Southern California, Los Angeles, CA 90089-2531. E-mail: trifunac@usc.edu
1. INTRODUCTION

Most of the structural health-monitoring methods for civil engineering structures are physically based on detecting changes in the modal parameters (frequencies and mode shapes) of the structure, which are global properties, changing little when the damage is localized. They are also sensitive to environmental influences (e.g. temperature) and changes in the boundary conditions (e.g. soil-foundation system), which are difficult to separate, and which may produce similar effects as damage on the recorded response (see Chang et al., 2003, and Doebling et al., 1996, for detailed state of the art reviews on this topic). Despite these known difficulties, the modal methods are still prevailing in structural health monitoring, and most of the recent developments in this field have been in the sensing, data transmission, and computational aspects of the problem. In contrast, this paper deals with the physical aspect of the problem. In particular, it presents an exploratory analysis of a new method for structural health monitoring, based on the wave propagation, rather than vibrational, view of the seismic response of structures. Alternatively to mode superposition, the seismic response of a structure can be represented as a superposition of waves that propagate through the structure, reflect from its exterior and interior boundaries and interfere (Kanai 1965; Snieder and Şafak 2006; Todorovska et al. 2001a,b). Loss of stiffness due to local damage would cause delays in the wave propagation through the damaged part, which could be detected using seismic response data recorded on each side of the damaged area, along the wave path. A change in wave travel time would depend only on the changes of the physical properties between the sensors. Hence the wave methods should be more sensitive to local damage, which would be a major advantage over the modal (vibrational) methods. Additionally, the local changes in travel time should be less sensitive to the effects of soil-structure interaction, which is a major obstacle for the modal
methods based on detecting changes in the structural frequencies.

This paper explores a new structural health monitoring method, based on detecting changes in travel times of seismic waves propagating through the structure using impulse response functions. The impulse response functions are computed by deconvolution of recorded seismic response at different levels in the structure, assuming vertically propagating waves. The method is tested on actual seismic response data recorded in a building that was damaged during the recorded earthquake shaking. The small amplitude response data within the initial time window are used as baseline. The test bed structure is the former Imperial County Services (ICS) Building – a 6-story reinforced concrete structure in El Centro, California – severely damaged by the Imperial Valley earthquake of October 15, 1979 ($M_L = 6.6$, depth $H=8$ km), and later demolished. The closest distance from the rupture to the building was only 7 km. The structure was instrumented by a 13 channel accelerograph array, and a free-field site, which all recorded the earthquake. The ICS building is a rare case of an instrumented building severely damaged by an earthquake.

This paper is the third in a sequel of applications of different structural health monitoring methods to this building by the same authors. The first paper (Todorovska and Trifunac 2006a) presented an analysis of the changes in the first soil-structure system frequency with time, and of the inter-story drifts estimated from the strong motion data. It also presented a summary of the description of the building, observed damage following an inspection after the earthquake (Kojić et al, 1984), and the recorded strong motion data. The second paper (Todorovska and Trifunac 2006b) presented an analysis of novelties (abrupt changes) in the recorded seismic response using expansion in a basis of bi-orthogonal wavelets.
The method presented in this paper differs from the wave methods used in non-destructive testing (NDT) of materials in that the latter typically use: (1) ultrasonic waves, which are attenuated quickly along the wave path, (2) need an actuator to create such waves, and (3) detect cracks, or some other defect in a member, using reflected waves from the defects. These methods are typically used locally, to detect the location of a defect in a member, but are impractical and too costly for global structural health monitoring (Chang et al. 2003). The method in this paper uses seismic waves, which are long (5-500 m) and are not much attenuated, does not need actuators, and is based on measurements of travel times of waves transmitted through the damaged zone.

There have been only a few publications in literature on wave propagation methods, other than NDT, for structural health monitoring and damage detection in civil structures (Şafak 1998; Ivanović et al. 2001; Trifunac et al. 2003; and Ma and Pines 2003). Şafak (1998) proposed a layered continuous model for analysis of seismic response of a building, and detection of damage by tracing changes in the parameters in the different layers. Ivanović et al. (2001) and Trifunac et al. (2003) used strong motion data recorded in a 7-story RC building in Van Nuys during the 1994 Northridge earthquake, to explore two methods, one based on cross-correlation analysis (to estimate time lags between motions recorded at different levels), and the other one based on detecting changes in wave numbers (inversely proportional to the wave velocities) of waves propagating between different levels. Ma and Pines (2003) proposed a method based on a lumped mass building model, and propagation of dereverberated waves to identify the damage, which they tested on simulated building response data.

Impulse response functions, computed by deconvolution from small amplitude seismic response, have been used by Snieder and Şafak (2006) to analyze Millikan Library, and also in
the analyses of geophysical and geotechnical borehole data (Haddadi and Kawakami 1998). To the knowledge of the authors, this paper is the first to show an application of such impulse response functions to detection of seismic damage in a structure.

2. METHODOLOGY

2.1 1D Continuous Wave Propagation Model of a Building

The nature of wave motion is such that long enough waves see a discrete medium they propagate through as a “continuum.” In fact, there is no such thing as continuum literally, as at the finest scale, all materials are discrete, i.e. made of atoms, which can further be decomposed into elementary particles. Hence, the seismic waves that are much longer than the dimensions of the discontinuities in a building would see it as a continuum. The simplest continuous model of a narrow building is a shear beam (Kanai 1965), and that of a long building is a shear plate (Todorovska and Trifunac 1989). If the mass and stiffness of the individual stories varies, then horizontally layered models, with piecewise continuous properties can be used, with interfaces at the floor slabs (Todorovska and Trifunac 1990; Todorovska and Lee 1989; Şafak 1999). Within a layer that is “homogeneous”, the wave paths will be straight lines, and at an interface between two different media, an incident wave will split into a reflected and a refracted wave. At the stress free boundaries, a wave is totally reflected. Deviation from a straight line of propagation (i.e. diffraction) will occur if there are inhomogeneities along the wave path with size comparable to the wavelength of the propagating wave. Finally, such inhomogeneities would appear as infinite barriers to very short waves (Todorovska and Lee 1989).

In a narrow building, deforming mainly in shear, the deformation due to horizontal earthquake ground motion can be modeled as one-dimensional (1D) wave propagation. The
same holds for a long regular building if the disturbance is homogeneous horizontally. The simplest such model is that of an equivalent uniform shear beam. Such a beam is traversed by infinitely many trains of waves, propagating upward and downward, each resulting from a direct incidence, or from a different generation reflection from the stress free boundary at the top, or from the interface with the ground at the bottom. Asymptotic formulae for the infinite sums are given in Kanai (1965) for the undamped case, and in Shieder and Şafak (2006) for the damped case. Further, Shieder and Şafak (2006) present a proof of the equivalence of such wave superposition representation of the response with the mode superposition representation. In a layered model, the representation would be more complex, due to reflections and refractions of each wave at each one of the interfaces, with the reflection and transmission coefficients depending on the impedance contrast. This is schematically illustrated in Fig. 1, where the ground is represented as another layer.

An input wave at the base will propagate upward and will be seen *delayed* and attenuated (due to material damping and reverberations due to reflections from the layer boundaries) at observation points at different heights along the building. At the top, it will be reflected back, and will be seen delayed at consecutive observation points down towards the base. After hitting the base, it will be partially reflected and will again propagate upwards. After many such reflections, the motion resulting from constructive interference will dominate the response. The time delay between the motions at different stories can be observed by a necked eye in some earthquake records in tall buildings, but, to measure such delays, it is more convenient to use some signal processing tool, the most common one being *cross-correlation* analysis. In this paper we use *deconvolution* analysis, as follows.
Let us view the building as a linear time-invariant system, with a single input – the ground motion, \( u_{\text{ref}}(t) \), and multiple outputs – the story responses, \( u_i(t) \). The input and outputs are related in the time domain by

\[
  u_i(t) = (u_{\text{ref}} * h_i)(t) = \int_{0}^{t} u_{\text{ref}}(\tau) h_i(t - \tau) d\tau
\]

and in the frequency domain by

\[
  \hat{u}(\omega) = \hat{u}_{\text{ref}}(\omega) \hat{h}_i(\omega)
\]

where \(*\) indicates convolution, and the hat indicates Fourier transform. Function \( \hat{h}_i(\omega) \) is called impulse response function, and represents the response at level \( i \) to input that is a Dirac delta function, \( \delta(t) \), that is

\[
  u_{\text{ref}}(t) = \delta(t) \iff u_i(t) = h_i(t)
\]

Function \( \hat{h}_i(\omega) \) is the transfer function between the response at level \( i \) and the input, and represents the Fourier transform of the response to input such that \( \hat{u}_{\text{ref}}(\omega) = 1 \). The transfer function is the Fourier transform of the impulse response function

\[
  \hat{h}_i(\omega) = \text{FT}\{h_i(t)\}
\]

Hence, the impulse response functions can be computed from any recorded response, by taking inverse Fourier transform of the corresponding transfer function, and can be conveniently used to
numerically simulate the propagation of a pulse through the building, using actual data. The time delays then can be measured using these impulse response functions. We note here that the response at any level can be used as reference motion, in which case the impulse response function for that level would be a Delta-function.

Practically, $h_i(t)$ can be computed using

$$h_i(t) = F T^{-1} \left[ \frac{\hat{u}_i(\omega) \hat{u}_{ref}(\omega)}{\hat{u}_{ref}(\omega) + \epsilon} \right]$$

(5)

where the bar indicates complex conjugate, $\epsilon$ is a regularization parameter used to avoid singularities (Snieder and Şafak 2006). In this paper, we used $\epsilon = 0.1*P$ when $u_{ref}$ is the ground floor record, and $\epsilon = 0.05*P$ when $u_{ref}$ is the roof record, where $P$ is the average power of $u_{ref}$.

2.2 Damage Detection

To identify damage by detecting changes in travel times, some reference travel times are needed to serve as baseline. For continuously monitored buildings, those could be values obtained from weak motion data recorded “immediately” before the earthquake, which can then be compared with values obtained from similar amplitude motions recorded after the earthquake. Also, using recorded strong motion data one could estimate “instantaneous” travel times from windowed data and track its changes versus time. In this exploratory analysis, we use strong motion data from an earthquake that damaged the building, and three time windows – before, during and after the occurrence of the major damage. The limits of these time intervals are chosen based on results of an analysis of novelties in the recorded acceleration response
(Todorovska and Trifunac 2006b). In each window, the analysis would give the properties of an equivalent linear system representing the building in the corresponding time window.

3. RESULTS AND ANALYSIS

3.1 The Imperial County Services Building

The former Imperial County Services (ICS) Building was a 6-story instrumented reinforced concrete structure in El Centro, California. Here we briefly describe the building, instrumentation layout, and the observed damage following the Imperial Valley earthquake of October 15, 1979 ($M_L = 6.6$, depth $H=8$ km), for convenience and completeness of this presentation. Further details about the design, recorded data and observed damage can be found in Kojić et al. (1984).

Figure 2 shows a photo of a side view of the building, and Fig. 3 shows the foundation (top) and a typical floor (bottom) layouts. The ground floor was 41.70 m (136 feet 10 inches) by 26.02 m (85 feet 4 inches) in plan, and the height of the building was 25.48 m (83 feet 7 inches). The foundation consisted of pile caps resting on Raymond tapered piles which were interconnected by grade beams (Fig. 3 top). Lateral resistance of all levels in the longitudinal (EW) direction was provided by two exterior moment frames at column lines 1 and 4, and two interior moment frames on column lines 2 and 3 (Fig. 3). The lateral resistance in the transverse (NS) direction was not continuous. At the ground floor level, it was provided by four short shear walls located along column lines A, C, D, and E and extending between column lines 2 and 3 only (Fig. 3 top). At the second floor and above, lateral (NS) resistance was provided by two shear walls at the east and west ends of the building (Fig. 3 bottom). This caused the building to appear top heavy with a soft first story (Fig. 2). The irregularities in the NS stiffness at the
ground floor appear to have resulted in excessive torsional response and in significant coupling of the NS and torsional excitations and responses. The design strength of the concrete was 34.5 MPa (5 ksi) for columns, 20.7 MPa (3 ksi) for the elements below ground level, and 27.6 MPa (4 ksi) everywhere else. All reinforcing steel was specified to be grade 40 ($F_y = 276$ MPa).

The building was instrumented by a 13 channel accelerograph array, and a free-field site, which all recorded the earthquake. Figure 4 shows a sketch of the building with the location of the sensors. The film records were digitized and released as 22.5 s of acceleration data equally sampled at 0.02 s, band-pass filtered with Ormsby filters between 0.1-0.125 and 25-27 Hz (see Fig. 5 in Todorovska and Trifunac 2006a). The recorded peak accelerations at the roof and ground floor were 571 cm/s$^2$ and 339 cm/s$^2$ in the NS direction and 461 cm/s$^2$ and 331 cm/s$^2$ in the EW direction. For this analysis, we further high-pass filtered the data at 0.2-0.3 Hz using also an Ormsby filter, and upsamples (by linear interpolation) to 0.01 s.

The building was severely damaged by this earthquake, and was later demolished (Kojić et al., 1984). Figure 5 shows a schematic representation of the observed damage. The major failure occurred in the columns of frame F (at the east end of the building) at the ground floor. The vertical reinforcement was exposed and buckled, and the core concrete could not be contained, resulting in sudden failure and shortening of the columns subjected to excessive axial loads. This in turn caused an incipient vertical fall of the eastern end of the building, causing cracking of the floor beams and slabs near column line E on the second, third and higher floors. Figure 6a shows a photo of the damage of columns F1 and F2 at the ground floor, and Fig. 6b shows a closer view of the buckled steel bars of column F1. Columns in lines A, B, D and E also suffered damage. Columns in frames A and E did not suffer as extensive damage as shortening
and buckling of the reinforcement in line F at the east side, but large concrete cracks and exposed reinforcement could be seen near the base. In the columns in interior frames B through E, visible cracks and spalling of the concrete cover were also observed (Kojić et al., 1984).

### 3.2 Impulse Responses and Travel Times for EW Motions

The analysis of novelties in the response of this building (Todorovska and Trifunac 2006b) suggested that damage first occurred at about 6.4 s, proceeded between 8.2 and 9.2 s, and culminated at about 11.2 s, with the collapse of the first story columns (at line F) at the east side of the building (Figs. 3, 5, and 6a,b). Based on these results, in this paper, we consider three time windows – before, during, and after the major damage occurred: $t < 7$ s, $7 < t < 13$ s, and $t > 13$ s. We measure the travel times of the identified pulses for each time window, and analyze the changes relative to the first time window, which we chose to serve as baseline data.

We first show results for the EW (longitudinal) response of the ICS building, for which a one dimensional shear wave propagation model is perhaps most appropriate, as the distribution of stiffness in the NS direction was nonsymmetrical, and the contribution of torsion to the recorded NS response was significant (Todorovska and Trifunac 2006a). EW motions were recorded at the ground floor (Channel 13), 2nd floor (Channel 6), 4th floor (Channel 5) and roof (Channel 4) (Fig. 4). Figure 7 shows impulse response functions for EW motions at different levels in the building, for an input impulse applied at the base (left), and at the roof (right). The different types of lines show results for different time windows. The impulse response functions are shown only for the early stages of response to emphasize the differences in arrival times of the pulses (for the different time windows, and at different levels in the building).
The plots on the left hand side in Fig. 7 show an impulse at time $t = 0$ in the ground floor motion, which propagates up, reaching the upper floors with some time delay, reflects from the roof, and propagates down. The “input” pulse has finite width due to the finite length of the records, and the regularization parameter $\varepsilon$. The downward propagating pulse is not seen at the ground floor due to the nature of eqn (5), which always gives an impulse at time $t = 0$, at the level used as reference. Within the time limits of the plots, its reflection from the ground floor can be seen only in the 2nd floor impulse response computed from the data from the first time window ($t < 7$ s).

The plots on the right hand side in Fig. 7 show an input pulse at time $t = 0$ at roof level, which propagates down with increasing time. Another acausal wave can be seen propagating down in negative time, which corresponds to the upward propagating waves in the physical model. This acausal wave confirms that no matter at which level the “input” pulse is introduced by the data processing, the results reveal the nature of the physical process, which consists of input from the base and total reflection from the roof.

The plots in Fig. 7 show that the travel times of the pulses for the three time windows are different, indicating longer travel times (i.e. reduced stiffness) within the second and third time window ($t > 7$ s) as compared to the first time window ($t < 7$ s). For this exploratory analysis of the capability of this method to detect damage in real data, we measured the arrival times from the plots of the impulse response functions by visual inspection. The values read are therefore approximate (we estimate the accuracy of reading the arrival times to be about 0.01 s). The travel times can be estimated more accurately by fitting a model to the data, but this is beyond the scope of this paper. The results are tabulated in Table 1. The upper part of this table shows the measurements for an input impulse at the base propagating up (Fig. 7 left), while the lower
part – for an input impulse at the roof propagating acausally down (in negative time) (Fig. 7 right). The first column shows: (1) the floor level where the acceleration sensor was located. The following group of two columns shows for $t < 7$ s: (2a) the arrival time, $t_i$, of the pulse at the particular sensor and (2b) the travel time $\tau_i$ of the pulse within segment $i$ of the building, consisting of the stories between two sensors. The following two groups of columns (3a,b and 4a,b) show the same quantities, respectively for $7 < t < 13$ s, and for $t > 13$ s. It can be seen from this table that the travel times obtained from the two signal processing experiments are mutually consistent, differing by about $10^{-2}$ s (which is 5.5% of the travel time through the entire height of the building above ground level). It can also be seen that the travel times increased during the second and third time window, relative to the first one, which indicates softening of the structure, consistent with the occurrence of damage.

### 3.3 Impulse Responses and Travel Times for NS Motions

The NS (transverse) motions were recorded at the ground floor, 2nd floor and roof, along the west side, center and east side of the building. An exception is the ground floor where there were only two sensors, one at the west side, and the other one between the center and east side of the building (Fig 4). We computed impulse response functions for the NS motions and measured travel times between sensors along each one of the three vertical arrays, and within each one of the three time windows. The results are presented in a similar fashion as for the EW motions (Fig. 7). Figures 8, 9 and 10 show impulse response functions respectively for NS motions recorded along the west side (channels 10, 7 and 1, see Fig. 4), the center (channels 11, 8 and 2), and the east side (channels 11, 9 and 3) of the building. The pulse arrival times and travel times between sensors are tabulated in Table 2, which show the same quantities as Table 1. As for the
EW motions, the travel times measured from the two signal processing experiments are not only mutually consistent, but are practically identical up to the accuracy of readings \(10^{-2} \text{ s}\).

### 3.4 Mean Travel Times and Inferences on Wave Velocities and Changes in Floor Stiffness

The mean travel times, \(\tau_i\) (over the two measurements, see Tables 1 and 2), were calculated and used in the subsequent analysis. The results are tabulated in Table 3 for EW motions and in Table 4 for NS motions. The first two columns in these tables show: (1) the floor level, and (2) the distance between neighboring sensors, \(h_i\). The following group of three columns shows for \(t < 7 \text{ s}\): (3a) the mean travel time \(\tau_i\), (3b) \(4\tau_i\) (= contributions from section \(i\) to the fundamental period of vibration of an equivalent uniform shear beam model of the building), and (3c) the average (over the floors within segment \(i\)) velocity of wave propagation, \(v_i = h_i / \tau_i\). The values of \(v_i\) for this particular time interval were used as baseline in analyzing the changes. The next two groups of five columns correspond respectively to time windows \(7 < t < 13 \text{ s}\), and \(t > 13 \text{ s}\). The first three columns in each group show the same quantities as columns (3a), (3b) and (3c). The additional two columns show the changes in: ((4d) and (5d)) the wave velocities, and ((4e) and (5e)) the rigidities relative to the baseline (the corresponding values for the first time interval, \(t < 7 \text{ s}\)). The change in velocities can be directly computed from the travel times as

\[
\Delta v_i / v_{ref} = (v_i - v_{ref}) / v_{ref} - 1.
\]

The change of rigidity was estimated based on the fact that, for almost uniform distribution of density along the height of the building, \(\mu_i \sim v_i^2\), where \(\mu_i\) is the shear modulus for segment \(i\) of the building. Then

\[
\Delta \mu_i / \mu_{ref} = \left(\frac{\tau_{ref}}{\tau_i}\right)^2 - 1.
\]

The results in Tables 3 and 4 are shown graphically in Figs 11, 12 and 13, and are discussed in the next two sections. First, the consistency of the initial estimates of the floor shear wave
velocities is examined. Then the changes in floor stiffness versus time, and their spatial
distribution are discussed.

3.4.1 Initial Shear Wave Velocities

Figure 11 shows schematically the spatial distribution of the “initial” (i.e. representative
values for time window $t < 7$ s) shear wave velocities, $v_i$, that are tabulated in Table 3. These
represent the average floor velocities between two sensors.

The results for EW motions (Table 3, Fig. 11 left) indicate initial wave velocities of 201 m/s
through the first floor, 183 m/s between the 2$^{nd}$ and 4$^{th}$ floors, and 111 m/s between the 4$^{th}$ floor
and roof. The velocity of an equivalent uniform shear bream is 142 m/s.

The results for NS motions at the west side of the building (Table 4 top, Fig. 11 right) imply
initial velocities of 168 m/s through the first floor, and 256 m/s through the upper part. The
higher velocity in the upper part is due to the end shear wall, which extended throughout the
entire width of the building, while the one at the first floor extended only along one third of the
width (see Fig. 3). The velocity of an equivalent uniform shear beam is 232 m/s.

The results for NS motions at the center of the building (Table 4 center, Fig. 11 right) imply
initial velocity of 503 m/s through the first floor, and 170 m/s in the upper part. The velocity of
an equivalent uniform shear beam is 196 m/s. The larger value for the first floor than for the
upper floors can be explained by the added stiffness from the three shear walls near the sensors,
which extended only over the height of the first floor, while there was no shear wall in the upper
part of the building between the sensors.
The results for NS motions at the east end of the building (Table 4 bottom, Fig. 11 right) imply initial velocity of 252 m/s through the first floor, and 186 m/s in the upper part. This is opposite of what one would expect, that is lower velocity through the first floor compared to the upper floors, considering the actual distribution of stiffness (Fig. 3). However, we note that the sensor at the ground floor was not at the east end, but about half way towards the center, near the last first floor shear wall. Hence, the “first floor velocity” obtained from the travel times is not representative of waves propagating up through the east side, but probably represents waves reaching the 2nd floor through the shear wall near sensor 8 and then propagating horizontally through the “rigid” floor slab. The lower than expected velocity for the upper part of the building is discussed the next paragraph.

Next we compare the initial NS velocities horizontally, examining their variations along the building length. For the upper part, the design drawings show that the actual “stiffness at the center” was smaller than at the ends, and that the stiffness at both ends was very similar. Hence, one would expect smaller velocity at the center and approximately equal velocities at the ends. We also know from the analysis of drifts that they were the smallest at the center and the largest at the east end (see Fig. 6 in Todorovska and Trifunac 2006a), even within the initial time window (\( t < 7 \text{ s} \)), before the severe damage occurred, as a result of the asymmetric distribution of stiffness at the first floor. The wave travel times give the smallest velocity at the center, as expected, but somewhat smaller velocity at the east side (186 m/s) than at the west side (256 m/s), which is consistent with the larger drifts at the east end, resulting in a “softer” structure even before the severe damage occurred. If true, the latter suggests strong dependence of the travel times (and inferred velocities), on the level of strain in the structural members.
In the lower part of the building, the design drawings suggest the smallest stiffness at the east end and the largest stiffness at the center, and the analysis of drifts shows largest drifts at the east end and smallest drifts at the center. The travel times give (by far) the largest velocity at the center (503 m/s as compared to 168 m/s at the west end), which is consistent with both the designed stiffness and the observed drifts. The travel times give velocity at the east end (252 m/s) larger than that at the west end, contrary to the expectations, but, as mentioned earlier, the value for the east end may not be representative of waves propagating through the columns of at the east side.

3.4.2 Changes in Stiffness

An most important check of the method is that it depicts correctly the changes in stiffness in the different parts of the structure. Figure 12 shows the spatial distribution of the changes in stiffness, $\Delta \mu / \mu_{\text{ref}}$, computed from the measured changes in travel times, and listed in Tables 3 and 4. The percentages shown represent the changes between the average stiffness in the third time window ($t > 12$ s) relative to the average stiffness in the first time window ($t < 7$ s). The changes for the EW motions suggest hardening in the 1st floor (by about 10%) during the third time window, following the large (69%) reduction during the second time window. Hence, for this segment of the building, the second interval reduction (80%) is shown in brackets, as the maximum value. At the bottom of Fig. 12, the changes in the fundamental fixed-base frequencies, computed from the total travel times are shown and the implied change of the overall stiffness.

Figure 12 suggests reduction of stiffness throughout the building between 40 and 80%. For EW motions, the reduction was the largest in the first story (80% during the second time
window), but was also large in the upper stories (72% between the 2nd and 4th floors, and 60% between the 4th floor and roof). This is consistent with the spatial distribution of the observed damage (Fig. 5), which was the largest in the first story. For NS motions, the reduction is significantly larger at the east side of the building than at the west side and center, both in the first story (75% as compared to 44% at the west side and 0% at the center) and above (63% as compared to 47% at the west side and 43% at the center), which is consistent with the spatial distribution of the observed damage (Fig. 5). One discrepancy between the observed damage and the reduction of stiffness estimated from travel times is that the former indicates no major cracks in the upper part along the west side and center of the building, while the latter suggests significant reduction of stiffness (47% and 43%). This discrepancy is likely due to damage in the upper part that was not easily visible and hence was not recorded.

Next, we examine the reduction of floor stiffness versus time. Figure 13 shows the floor stiffness, as a fraction of the initial stiffness, for EW motions (top) and NS motions (bottom). The curves represent linear interpolation between the representative interval values, assigned to the central time of the window. We note here that the interval values are weighted averages of the instantaneous values within the time window. Hence, damage that occurred near the end of the second time window would not contribute much to the weighted average for that window, but would reflect on the interval value for the third time window. In view of this fact, the changes in stiffness in Fig. 13 suggest that, most of the reduction of stiffness in the EW direction started to occur, or occurred relatively early within the time window. The same is true for the reduction of stiffness in the NS direction everywhere in the upper part of the building, while in the first story, at the both ends of the building, the NS stiffness changed significantly near the end of the second time window, especially at the east end. This is consistent with the timing of the collapse of the
first story columns in row F, which occurred at the very end of the second time interval (at about 11.2 s; Todorovska and Trifunac 2006b), and affected mostly the first floor NS stiffness at the east side of the building. The first story NS stiffness did not change at the center of the building.

The two horizontal lines in Fig. 13 correspond to values obtained by Kojić et al. (1984) for two models using the readily available structural analysis software at that time – ETABS, which was linear. Due to this limitation of the software, they analyzed the three-dimensional nonlinear response of the building using equivalent linear elastic models. They accounted for the effects of the flexibility of the soil and the soil-structure-interaction, they adding a fictitious story beneath ground level. Their Model $II_A$ considered the full stiffness of all structural members, i.e. the initial state of the building, while their model $II_B$ considered reduced stiffness, to account for the effects of nonlinear structural response and damage. In particular, they reduced the moments of inertia of some columns at the ground level, but did not change their axial stiffness, and did not change the stiffness of the structural elements above the second floor. The reduction of the moments of inertia was 70% for the columns along lines A, B, E and F (see Fig. 4 top), and 20% for those along lines C and D. Further, they introduced hinge supports for columns E1, E4 and for all columns on the line F, at ground level. The resulting frequencies of vibration for Model $II_A$ were 0.91 Hz for EW motions, and 1.64 Hz for NS motions, and for Model $II_B$ they were 0.80 Hz for EW motions and 1.61 Hz for NS motions, all shown, by horizontal lines in Fig. 14. The reductions of stiffness in the first story columns by 20 or 70 percent are also indicated in Fig. 13 by wide horizontal lines. The qualitative agreement of the characteristics of the ETABS model chosen by Kojić et al. (1984) with the results of the present study is excellent.
3.5 Fundamental Fixed-Base Frequency Estimates from Wave Travel Times

The changes in travel times from one time window to another can be cast into changes in the interference conditions, which relate to the fundamental fixed-base period of vibration of a shear beam, $T_1$, and the corresponding frequency $f_1 = 1/T_1$. Assuming a one-dimensional (1D) model and deformation in shear only, the height of the building above ground level is $1/4$ of the wavelength, which gives

$$T_1 = 4\tau_{\text{tot}}$$

where $\tau_{\text{tot}}$ is the time it takes for the input impulse to traverse the height of the building. Also, as $\tau_{\text{tot}} = \sum_i \tau_i$, $T_i = \sum_i (4\tau_i)$, and $f_i = 1/\sum_i (4\tau_i)$. We note here that, because of the assumption that the motion of the building consists of vertically propagating waves, due to shear deformation only, the estimated values of $f_i$ from travel times are lower bounds.

We computed $T_i$ and $f_i$ from travel times using eqn (6) for EW motions, for which a 1D wave propagation model appears to be appropriate. We also computed $T_i$ and $f_i$ from the NS motions travel times for the west side, center and the east side of the building. The NS response was clearly two dimensional and asymmetric. Hence, most meaningful of these frequencies is $f_i$ for the center of the building, as the closest one to the fundamental fixed-base frequency for NS translation, while we use the other two values, for the sides of the building merely as measures of the stiffness of the building at both sides. The values for $\tau_{\text{tot}}$, $T_i$ and $f_i$ are summarized in Table 5 for the three time windows. For the second and third time windows, the percentage change in frequency and in rigidity of a corresponding 1D equivalent uniform shear beam are
It can be seen that the overall EW stiffness of the structure was reduced by 65%. The overall NS stiffness was reduced by 46% at the west side of the building, by 41% at the center, and by 65% at the east side. The largest decrease at the east side of the building, as suggested by the analysis of travel times, is consistent with the observed damage. The initial values of $f_1$ and the implied changes in the overall stiffness are shown also in the sketches in Fig. 11 and 12.

Figure 14 shows graphically the changes of $f_1$ with time for EW (top) and NS (bottom) motions, and compares $f_1$ from travel times with other estimates of $f_1$ and of the soil-structure system frequency, $f_{sys}$. The open circles show $f_1$ from travel times at the center of the corresponding time interval, and the line connecting the circles represents interpolated values. The other estimates include: (1) the system frequencies, $f_{sys}$, measured from ambient vibration tests conducted before the Imperial Valley earthquake (Pardoen 1979), (2) “instantaneous” system frequency $f_{sys}$ estimated from the recorded response to the 1979 Imperial Valley earthquake using Gabor transform (Todorovska and Trifunac 2006a), (3) “first mode” frequencies of the soil-structure system $f_{1}^{*}$ estimated from two equivalent linear ETABS models, $II_A$ and $II_B$, (shown by horizontal lines; Kojić et al. 1984), and (4) “instantaneous” fixed-base frequency $f_1$ estimated from travel times of high frequency pulses using decomposition of the response to the Imperial Valley earthquake in a wavelet basis (shown by solid dots; Todorovska and Trifunac 2006b). The vertical lines, T1, T2 and T3, show the times when major damage occurred, as estimated from the analysis of novelties in the recorded response to the Imperial Valley earthquake (Todorovska and Trifunac 2006b).
The system frequencies, $f_{sys}$, measured by ambient vibration tests (Pardoen 1979) are shown along the left edge of the plots (by a short hatched line). These values are 1.54 Hz for EW motions, 2.24 Hz for NS motions at the center of the building, and 2.81 Hz for NS motions at the west side, which was interpreted as the first torsional frequency. It is noted here that the ambient vibration measurements (Pardoen 1979) were made without using a reference site. Consequently, the measured structural frequencies may not be accurate due to influence from the site frequencies.

We first examine the consistency of the different estimates of frequency for EW motions. It can be seen from Fig. 14 (top) that the interpolated interval estimates of $f_1$ during the earthquake are always higher than the moving window estimates of $f_{sys}$ during the earthquake. This is as would be expected, in view of the fact that the two frequencies are related by

$$f_{sys}^{-2} = f_1^{-2} + f_H^{-2} + f_R^{-2}$$

(7)

where $f_H$ and $f_R$ represent the horizontal and rocking frequencies of a rigid building on flexible soil, which implies that $f_{sys}$ is smaller than the smallest of $f_1$, $f_H$ or $f_R$. The value of $f_{sys}$ during the ambient vibration tests is higher not only than $f_{sys}$ during the earthquake shaking, but also higher than $f_1$ during the earthquake shaking, which can be explained by the very small amplitudes of the response, excited by cultural noise and wind.

Next we examine the relationship between the interpolated interval estimates of $f_1$ from impulse responses travel times and the instantaneous estimates of $f_1$ from travel times of high frequency pulses identified in the analysis of novelties (Todorovska and Trifunac 2006b). It can
be seen that for all of the identified pulses, the *instantaneous* estimates of $f_1$ are lower than the *interval* estimates of $f_1$, but are still higher than the moving window estimates of $f_{sys}$, except for pulse g3. The lower *instantaneous* $f_1$ from novelties could be explained by the fact that (a) these values are representative of very short time intervals, and hence exhibit larger fluctuations than the smoother *interval* values, which are averages over much longer time windows, and (b) they were typically measured during extreme drift amplitudes (e.g., pulse g3 occurred at a time of about 1% first story drift). Events T1, T2, and T3 were associated with the times of major damage (Todorovska and Trifunac 2006b). The time of the significant fall in the values of $f_1$ are consistent with the times of T1, T2, and T3.

Similar observations can be made for the different estimates of the frequencies for NS motions, shown in Fig. 14 (bottom). Due to the larger stiffness at the west side of the building, relative to the center (additional shear wall between the 2nd floor and roof), the corresponding initial value of $f_1$ is larger than that for the center. The initial value of $f_1$ for the east side is also smaller than for the west side, due to the absence of a shear wall on line F at the ground floor (Fig. 3 top). The NS system frequency, which was estimated from the recorded earthquake response at the center, and is hence denoted by $f_{sys,Center}$, is always smaller than $f_{1,Center}$. It can also be seen that the system frequencies from ambient vibration tests measured from the response at the west side and at the center are higher than the corresponding initial values of $f_1$ during the earthquake shaking, which can be explained by the very small amplitudes of response to ambient noise.
4. SUMMARY AND CONCLUSIONS

This paper explored a new structural health monitoring method, based on detecting changes in the stiffness of the structural members by measuring changes in travel times of seismic waves propagating through these members. The travel times were measured from impulse response function computed from seismic response data recorded within three time windows – before, during and after the occurrence of the major damage, as determined by previous (high time resolution) analysis of novelties. The changes in travel times were measured relative to the initial time window. Hence this method did not require baseline data measured before the earthquake, or prediction of the response for different damage scenarios, using numerical simulations. A further and major advantage of this method is that it is local, as opposed to the modal methods, which are global, and hence can point out to the location of the damage with data from fewer sensors than the methods that detect changes in mode shapes. In this paper, this method is examined using real earthquake response data recorded in a full-scale building damaged by the earthquake.

The method is presented and applied in this paper in its most rudimentary form, based on several assumptions. One assumption is that one-dimensional wave propagation up and down the structure can capture the principal features of the response, and that side reflections of the non-vertically propagating waves (Todorovska et al. 1988) can be neglected. Another assumption is that it is sufficient to work only with the recorded horizontal translations. Further, it is assumed that the wave propagation effects associated with the incident seismic waves, through the foundation, horizontally can be neglected (Gičev and Trifunac, 2006; Trifunac et al. 1999), that the structural response resulting from warping and deformation of the foundation can be neglected (Gičev, 2005), and that the rotational waves in the building, caused by soil-
structure-interaction, can also be neglected. Furthermore, we did not consider explicitly the
detailed nature of the contributions of torsion and of rocking to the recorded horizontal NS
translations.

The results showed that, despite the simplifying assumptions, even for time windows as short
as 5 s, the method yielded meaningful impulse responses and wave travel times between sensors.
The inferred spatial distribution of the initial shear wave velocities throughout the building was
found to be consistent with the actual distribution of stiffness, and the spatial distribution of the
changes in stiffness was found to be generally consistent with the distribution of the observed
damage. Further, the fundamental fixed-base frequencies and their changes were estimated from
the measured travel times, and were found to be consistent with other estimates of frequency,
such as the soil-structure system frequency during the earthquake estimated from the recorded
motions, and by ETABS models, and the soil-structure system frequency estimated from
ambient vibration tests data.

We conclude that the analysis of wave travel times in a building undergoing damaging
response via impulse response functions, computed from the recorded seismic response, can
provide useful information about the degree and spatial distribution of the changes in the
component stiffness. The spatial resolution of this method depends on the number and the
separation distance of the recording instruments, and its temporal resolution would dependent on
the length of the time window chosen for the analysis. This method can be a useful tool for
structural health monitoring, and therefore should be further improved and refined.
REFERENCES


20. Todorovska, MI, M.D. Trifunac (2006a). Damage detection in the Imperial County Services Building I: the data and time-frequency analysis, submitted for publication.


Table 1  EW motions: measurements of arrival times, $t_i$, and of wave travel times, $\tau_i$ (from the plots in Fig. 7).

<table>
<thead>
<tr>
<th>EW motions</th>
<th>t &lt; 7 s</th>
<th>7 &lt; t &lt; 13 s</th>
<th>t &gt; 13 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) (2a) (2b) (3a) (3b) (4a) (4b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor</td>
<td>$t_i$</td>
<td>$\tau_i$</td>
<td>$t_i$</td>
</tr>
<tr>
<td>Roof</td>
<td>0.17</td>
<td>0.31</td>
<td>0.30</td>
</tr>
<tr>
<td>4th</td>
<td>0.07</td>
<td>0.14</td>
<td>0.13</td>
</tr>
<tr>
<td>2nd</td>
<td>0.03</td>
<td>0.05</td>
<td>0.04</td>
</tr>
<tr>
<td>Ground</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Table 2  NS motions recorded at the west side, center, and east side of the building: measurements of arrival times, $t_i$, and of wave travel times, $\tau_i$ (from the plots in Fig. 8, 9 and 10).

<table>
<thead>
<tr>
<th></th>
<th>NS motions</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$t &lt; 7$ s</td>
<td>$7 &lt; t &lt; 13$ s</td>
<td>$t &gt; 13$ s</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1)</td>
<td>(2a)</td>
<td>(2b)</td>
<td>(3a)</td>
</tr>
<tr>
<td>Floor</td>
<td>(t_i) s</td>
<td>(\tau_i) s</td>
<td>(t_i) s</td>
<td>(\tau_i) s</td>
</tr>
<tr>
<td>Roof</td>
<td>0.11</td>
<td>0.13</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>Ground</td>
<td>0.08</td>
<td>0.095</td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td>2nd</td>
<td>0.03</td>
<td>0.035</td>
<td>0.04</td>
<td></td>
</tr>
<tr>
<td>Ground</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td></td>
</tr>
</tbody>
</table>

West end

<table>
<thead>
<tr>
<th>Input impulse at</th>
<th>Ground, pulse going up</th>
<th>Ground, pulse going down</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2nd</td>
<td>0.08</td>
<td>-0.08</td>
</tr>
<tr>
<td>Ground</td>
<td>0.00</td>
<td>-0.10</td>
</tr>
</tbody>
</table>

Center

<table>
<thead>
<tr>
<th>Input impulse at</th>
<th>Ground, pulse going up</th>
<th>Ground, pulse going down</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.13</td>
<td>0.15</td>
</tr>
<tr>
<td>2nd</td>
<td>0.01</td>
<td>0.03</td>
</tr>
<tr>
<td>Ground</td>
<td>0.00</td>
<td>0.05</td>
</tr>
</tbody>
</table>

East end

<table>
<thead>
<tr>
<th>Input impulse at</th>
<th>Ground, pulse going up</th>
<th>Ground, pulse going down</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2nd</td>
<td>-0.11</td>
<td>-0.14</td>
</tr>
<tr>
<td>Ground</td>
<td>-0.13</td>
<td>-0.16</td>
</tr>
</tbody>
</table>
Table 3  EW motions: mean travel times $\tau_i$ (over the two measurements; see top and bottom parts of Table 1), average wave velocities for the segments of the building between sensors, $v_i$, and percent changes in these velocities and corresponding rigidities (relative to time interval $t < 7$ s).

<table>
<thead>
<tr>
<th>Floor</th>
<th>$h_i$ - m</th>
<th>$\tau_i$ - s (mean)</th>
<th>$4\tau_i$ - s</th>
<th>$\bar{v}<em>i$ - m/s = $v</em>{ref}$</th>
<th>$4\bar{v}_i$ - s</th>
<th>$\Delta \bar{v}<em>i / v</em>{ref}$ %</th>
<th>$\Delta \mu_i / \mu_{ref}$ %</th>
<th>$\tau_i$ - s (mean)</th>
<th>$4\tau_i$ - s</th>
<th>$\bar{v}_i$ - m/s</th>
<th>$\Delta \bar{v}<em>i / v</em>{ref}$ %</th>
<th>$\Delta \mu_i / \mu_{ref}$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>12.23</td>
<td>0.110</td>
<td>0.44</td>
<td>111</td>
<td>0.175</td>
<td>0.70</td>
<td>-37</td>
<td>-60</td>
<td>0.175</td>
<td>0.70</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td>4th</td>
<td>8.22</td>
<td>0.045</td>
<td>0.18</td>
<td>183</td>
<td>0.080</td>
<td>0.32</td>
<td>-44</td>
<td>-68</td>
<td>0.085</td>
<td>0.34</td>
<td>97</td>
<td>97</td>
</tr>
<tr>
<td>2nd</td>
<td>5.03</td>
<td>0.025</td>
<td>0.10</td>
<td>201</td>
<td>0.055</td>
<td>0.22</td>
<td>-55</td>
<td>-80</td>
<td>0.045</td>
<td>0.18</td>
<td>112</td>
<td>112</td>
</tr>
<tr>
<td>Ground</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4  NS motions recorded at the west side, center and east side of the building: mean travel times \( \tau_i \) (over the two measurements, see Table 2), average wave velocities for the segments of the building between sensors, \( v_i \), and percent changes in these velocities and corresponding rigidities (relative to time interval \( t < 7 \text{ s} \)).

<table>
<thead>
<tr>
<th>NS Motions at West end</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>( t &lt; 7 \text{ s} )</td>
</tr>
<tr>
<td>(1) (2) (3a) (3b) (3c) (4a) (4b) (4c) (4d) (4e) (5a) (5b) (5c) (5d) (5e)</td>
</tr>
<tr>
<td>Floor</td>
</tr>
<tr>
<td>Roof</td>
</tr>
<tr>
<td>2nd</td>
</tr>
<tr>
<td>Ground</td>
</tr>
<tr>
<td>Center</td>
</tr>
<tr>
<td>2nd</td>
</tr>
<tr>
<td>Ground</td>
</tr>
<tr>
<td>East end</td>
</tr>
<tr>
<td>2nd</td>
</tr>
<tr>
<td>Ground</td>
</tr>
</tbody>
</table>
Table 5  Equivalent shear wave velocities of a uniform shear beam, $v_{eq}$, and fundamental fixed-base frequencies of vibration, $f_1$, for the three time windows, estimated from mean wave travel times $\tau_{tot}$ over the building height ($f_1 = 1/(4\tau_{tot})$), for EW motions, and for NS motions recorded at the West end, center, and East end of the building. The percentage changes in $f_1$ and the equivalent rigidity are also shown.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Side</th>
<th>$t &lt; 7 \text{ s}$</th>
<th>$7 &lt; t &lt; 13 \text{ s}$</th>
<th>$t &gt; 13 \text{ s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(1a) (1b) (1c)</td>
<td>(2a) (2b) (2c)</td>
<td>(2d) (2e)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\tau_{tot}$ s</td>
<td>$v_{eq}$ m/s</td>
<td>$f_1$ Hz</td>
</tr>
<tr>
<td>EW</td>
<td>Center</td>
<td>0.18 142 1.39</td>
<td>0.31 82 0.81</td>
<td>-42 -66</td>
</tr>
<tr>
<td></td>
<td>West</td>
<td>0.11 232 2.27</td>
<td>0.13 196 1.92</td>
<td>-15 -28</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.13 196 1.92</td>
<td>0.15 170 1.67</td>
<td>-13 -24</td>
</tr>
<tr>
<td></td>
<td>East</td>
<td>0.13 196 1.92</td>
<td>0.16 159 1.61</td>
<td>-16 -30</td>
</tr>
<tr>
<td>NS</td>
<td>West</td>
<td>0.15 170 1.67</td>
<td>0.17 150 1.47</td>
<td>-23 -41</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.22 116 1.14</td>
<td>0.22 116 1.14</td>
<td>-41 -65</td>
</tr>
</tbody>
</table>
Figure Captions

Fig. 1  A three layer building model.

Fig. 2  General View (towards North) of Imperial County Services Building.

Fig. 3  Foundation and ground level plan (top) and typical floor layout (bottom) of the ICS building.

Fig. 4  Layout of the seismic monitoring array in the ICS building.

Fig. 5  Schematic representation of the damage in the ICS building following the 1979 Imperial Valley earthquake (reproduced from Kojić et al. (1984)).

Fig. 6  (a) Damage of columns F1 and F2 at the ground floor.  (b) Damage of column F1.

Fig. 7  Impulse response functions for EW motions and for input impulse at the ground floor (left) and at the roof (right).

Fig. 8  Same as Fig. 7 but for NS motions recorded at the west side of the building.

Fig. 9  Same as Fig. 7 but for NS motions recorded at the center of the building.

Fig. 10  Same as Fig. 7 but for NS motions recorded at the east side of the building.

Fig. 11  Initial \((t < 7 \text{ s})\) shear wave velocities, \(v_i\), for different segments of the building. At the bottom, the equivalent shear wave velocities for a uniform shear beam, \(v_{eq}\) are given, and the estimated fundamental fixed-base frequencies, \(f_1\).

Fig. 12  Final reductions of shear moduli, \(\mu_i\), for different segments of the building estimated from changes in travel times. At the bottom, the changes in \(f_1\) and the equivalent shear modulus are shown.

Fig. 13  Reduction of floor stiffness versus time.

Fig. 14  Frequency of vibration for EW (top) and NS (bottom) motions: comparison values from different measurements.
Fig. 1 A three layer building model.

Fig. 2 General View (towards North) of Imperial County Services Building.
Fig. 3  Foundation and ground level plan (top) and typical floor layout (bottom) of the ICS building.
Fig. 4 Layout of the seismic monitoring array in the ICS building.
Vertical reinforcement exposed and buckled. Concrete core was not retained.

- Shortening of line F columns caused cracking of floor beams and slabs between column lines E and F on the second, third and higher floors.
- Large concrete cracks, spalling of concrete cover and exposed reinforcement.
- Vertical reinforcement exposed and buckled. Concrete core was not retained.

Fig. 5 Schematic representation of the damage in the ICS building following the 1979 Imperial Valley earthquake (reproduced from Kojić et al. (1984)).

Fig. 6 (a) Damage of columns F1 and F2 at the ground floor. (b) Damage of column F1.
Fig. 7 Impulse response functions for EW motions and for input impulse at the ground floor (left) and at the roof (right).
Fig. 8  Same as Fig. 7 but for NS motions recorded at the west side of the building.

Fig. 9  Same as Fig. 7 but for NS motions recorded at the center of the building.
Fig. 10  Same as Fig. 7 but for NS motions recorded at the east side of the building.
Initial velocities $v_i$ ($t < 7$ s)

<table>
<thead>
<tr>
<th>EW motions</th>
<th>NS motions</th>
</tr>
</thead>
<tbody>
<tr>
<td>111 m/s</td>
<td>256 m/s</td>
</tr>
<tr>
<td>183 m/s</td>
<td>170 m/s</td>
</tr>
<tr>
<td>201 m/s</td>
<td>186 m/s</td>
</tr>
</tbody>
</table>

$\nu_{eq} = 142$ m/s  $\nu_{eq} = 232$ m/s  $\nu_{eq} = 196$ m/s  $\nu_{eq} = 196$ m/s

$f_1 = 1.39$ Hz  $f_1 = 2.27$ Hz  $f_1 = 1.92$ Hz  $f_1 = 1.92$ Hz

Fig. 11 Initial ($t < 7$ s) shear wave velocities, $v_i$, for different segments of the building. At the bottom, the equivalent shear wave velocities for a uniform shear beam, $\nu_{eq}$ are given, and the estimated fundamental fixed-base frequencies, $f_1$.

Reduction of shear modulus, $\mu$

<table>
<thead>
<tr>
<th>EW motions</th>
<th>NS motions</th>
</tr>
</thead>
<tbody>
<tr>
<td>60%</td>
<td>47%</td>
</tr>
<tr>
<td>72%</td>
<td>43%</td>
</tr>
<tr>
<td>69% (80%)</td>
<td>63%</td>
</tr>
</tbody>
</table>

$\nu$: 61%  $\nu$: 61%  $\nu$: 61%  $\nu$: 61%

Fig. 12 Final reductions of shear moduli, $\mu_i$, for different segments of the building estimated from changes in travel times. At the bottom, the changes in $f_1$ and the equivalent shear modulus are shown.
Fig. 13  Reduction of floor stiffness versus time.
Fig. 14 Frequency of vibration for EW (top) and NS (bottom) motions: comparison values from different measurements.