UNIVERSITY OF SOUTHERN CALIFORNIA
Department of Civil Engineering

INSTRUMENTED 7-STOREY REINFORCED CONCRETE BUILDING IN VAN NUYS, CALIFORNIA:
DESCRIPTION OF THE DAMAGE FROM THE 1994 NORTHRIIDGE EARTHQUAKE
AND STRONG MOTION DATA

by

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ABSTRACT

This report presents photographs and description of the damage in a seven-story reinforced concrete building in the city of Van Nuys in the Los Angeles metropolitan area caused by the Northridge, California, earthquake of 17 January, 1994. This earthquake was of moderate size ($M_L=6.4$), but occurred right beneath the densely populated San Fernando Valley and caused extensive damage to buildings and to the infrastructure. This building was located about 1.5 km east from the epicenter. It has been instrumented since 1967, and has recorded many earthquake and aftershocks, including the 1971 San Fernando, 1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear, and 1994 Northridge earthquakes. This report also describes strong-motion data from nine earthquakes and three aftershocks recorded in this building, up to December of 1994. It is intended to serve as supporting documentation for studies of the seismic response of this building and the damage caused by the 1994 Northridge earthquake. A brief description of studies of this building conducted by the authors is also included, to illustrate type of analyses these data can be used for.
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1. INTRODUCTION

1.1 Objectives

The Northridge earthquake \((M_L = 6.4, \text{ hypocenter at } 34^\circ 12.8' \text{ N, } 118^\circ 32.22' \text{ W and } 18 \text{ km depth}; [Wald et al., 1996])\) shook the Los Angeles metropolitan area in the early morning of January 17, 1994, and caused extensive damage to residential and commercial buildings and to the infrastructure [EERI, 1994]. It occurred on a blind thrust fault underneath San Fernando Valley, in the north-west part of the metropolitan area. Strong ground motion was recorded by more than 200 stations. The main event has been followed by hundreds of aftershocks, some with \(M \geq 5\) (a summary of processed strong ground motion records of the main event and of five \(M \geq 5\) aftershocks can be found in a report by Todorovska et al. [1999], and on the World Wide Web at http://ww.usc.edu/dept/civil_eng/Earthquake_eng/. Response of many instrumented buildings, some in the close proximity to the source, was also recorded.

One of the instrumented buildings that recorded the Northridge earthquake was a seven-story hotel, located ~1.5 km west from the epicenter. This building was a reinforced concrete structure, 62x150 feet (18.9x45.7 m) in plan, with essentially uniform distribution of mass and stiffness, and supported by concrete friction piles ~ 40 feet (12.2 m) long. The site geology is recent alluvium, consisting primarily of fine sandy silts and silty fine sands. This building was severely damaged by the Northridge earthquake and its early aftershocks, and was classified as unsafe (red-tagged) by the Los Angeles Department of Building Safety. Severe structural damage occurred in the exterior longitudinal frames (South and North), designed to take most of the lateral loads (in the EW direction). The nonstructural damage was also extensive. The peak accelerations recorded at the base of the buildings were 0.45g (L), 0.27g (V) and 0.40g (T). Figure 1.1 shows the location of the building relative to the causative fault and major freeway structures. The short dashed line shows the projection of the ruptured area and the long dashed lines show contours of peak horizontal velocity [Trifunac at al., 1996] inferred from strong motion recordings (the locations of the strong motion stations is indicated by the small circles, triangles, rectangles and diamonds). The large shaded area indicates rock and the white area indicates sediments. As background information, the locations of extensive damage to the freeway structures (the large solid circles), of zones of concentrated ground breakage (outlined by the irregular shaded symbols), and of other areas of interest in studies of the Northridge earthquake (marked as A, B, C and D) are also shown.

This report documents the damage of this building caused by the 1994 Northridge earthquake, based on information gathered by the USC team during two site visits, on
Figure 1.1 The location of the VN7SH in the epicentral area of the Northridge earthquake of 17 January, 1994 (M_L=6.4). The short dashed line outlines the surface projection of the fault. The light gray areas indicate bedrock, and the white areas—quaternary sediments. The small circular, triangular, square and diamond symbols indicate sites of strong motion stations. The hatched regions outlined by heavy solid lines indicate zones of concentrated ground breakage, and the open circles (1 to 6) locations of extensive damage or collapse of freeway structures. The heavy and long dashed lines show contours of horizontal peak velocity (in cm/s) of strong ground motion recorded in the free-field (Trifunac et al., 1996).
February 4 and April 19, 1994. The second visit took place following one of the larger aftershocks (of March 20, 1994, M=5.2) with epicenter ~1.2 km North of the building. This report also includes a brief description of the strong motion recordings in the building from nine earthquakes and three aftershocks between February of 1971 and December of 1994. This includes the 1994 Northridge main event and two aftershocks, as well as the 1971 San Fernando earthquake. The latter, with M=6.6 and epicenter ~22 km north-west from the building (Trifunac 1974), also damaged the building, but the structural damage was minor. The largest of the recorded motions are those of the 1994 Northridge earthquake. The instrumentation is operated by the Strong Motion Instrumentation Program of the California Division of Mines and Geology (CDMG).

This building will be referred to in this report as the VN7SH building. It is an interesting case-study because of several reasons: (1) it has been instrumented over a long period of time during which many earthquakes were recorded, (2) it was damaged by two of these earthquakes (1971 San Fernando and 1994 Northridge), and (3) ambient vibration measurements and analyses of the damaged building were conducted following both the 1971 San Fernando [Freeman and Honda, 1973] and the 1994 Northridge [Ivanović et. al., 1999] earthquakes. This report is intended to serve as reference documentation for studies of the seismic performance of this building, and of generic buildings with similar simple geometry [Trifunac and Todorovska, 2001].

1.2 Organization of this Report

Chapter 2 describes the building, and Chapter 3 describes the processed strong motion data recorded in the building. Chapter 4 describes the damage from the Northridge earthquake. It also includes a brief description of the damage from the 1971 San Fernando earthquake. Chapter 5 deals with design considerations related to the observed damage from the Northridge earthquake. It discusses the design strength capacity of the exterior frame columns and the loads corresponding to the maximum recorded drifts. The remaining part of the introduction summarizes several completed and ongoing studies of the building. The purpose is to illustrate, via examples, the value of full-scale experiments in describing the dynamic characteristic of real, three-dimensional structures. The results of these studies can be used as a guideline in installing new or upgrading existing instrumentation in buildings, that can provide more valuable information on the structural performance during earthquakes [Trifunac and Todorovska, 2001]. Appendices I and II show photographs of damage documented respectively on February 4, 1994, and on April 19, 1994.

In 2001, the list of references was updated for those items that got published in the meantime.
1.3 Review of Selected Studies of the VN7SH Building

1.3.1 Wave Propagation Analyses

Todorovska et al. [2001a,b] estimated the wave velocities in the VN7SH building from pairs of channels of recorded response to several earthquakes (via plots of wavenumbers of shear waves versus frequency). They obtained ~50–100 m/s for waves propagating vertically along the columns and ~500–2000 m/s for the waves propagating horizontally along the floor slabs. These values were consistent from one earthquake to another, prior to 1994, and in parts of the building not damaged by the 1994 Northridge earthquake. For waves propagating horizontally along the ground floor, these values were consistent with the apparent Love wave velocities in the soil. Trifunac et al. [2001c] repeated the same analysis for more earthquakes and including the damaged parts of the building. They concluded that, in the damaged parts, the phase velocities were consistent from one earthquake to another before the damage occurred, and differed after the damage occurred. This qualitative analysis concluded that wave propagation methods may be developed to detect the location of structural damage in a building, but to fully develop such methods more dense instrumentation is needed.

Ivanović et al. [2001] also analyzed the earthquake response data but using cross-correlation analysis with sliding time window. Their results indicated significant delays in wave travel times through those parts of the structure that were known to have experienced major damage from 1994 Northridge earthquake. This type of analysis is also a promising tool for structural health monitoring and for damage detection.

1.3.2 Spectral Characteristics of the Building-Soil Response

Trifunac et al. [2001a] analyzed strong motion data recorded in the building before and after the 1994 Northridge earthquake. They computed the ratio of Fourier spectrum of the relative response at the top with respect to the first (ground) floor (for a linear system, this ratio would be the transfer-function between the motion at the top and at the bottom), and found that it had a broad peak at frequency less than 1 Hz, lower than the fundamental fixed-base frequency estimated from the structural characteristics or from ambient vibration tests ($f > 1.4$ Hz for the NS vibrations and $f > 1–1.1$ Hz for the EW vibrations), indicating “loss of stiffness” of the soil-structure system during strong motion response. This “loss of stiffness” could not be explained by nonlinearity of the structure itself or by structural degradation due to damage only, leading to the interpretation that the nonlinearity of the soil contributed mainly to the nonlinear behavior of the soil-structure system.
1.3.3 Time-Frequency Analysis of the Building-Soil Response

The soil-structure interaction allows rocking and torsion of the building foundation. The motions in each of the two planes of symmetry of the building can be represented as a sum of the rocking motion of the buildings as a rigid body, and of the relative deflection due to lateral deformation of the columns. Ideally, for a rigid foundation, the NS rocking during strong earthquake shaking would be calculated from two vertical recordings at the opposite ends of the building. Unfortunately, this could not be done for the VN7SH building because of insufficient instrumentation (only one vertical sensor at ground level). In their study of the amplitude and time dependent changes in the period of the VN7SH building using all available strong motion data, Trifunac et al. [2001b] separated approximately the foundation “rocking” and the building relative response by band-pass filtering of the total system response. The following describes their findings.

The nonlinear effects in a time response of a system depend on the level of the excitation and on the initial state. In the case of a building-soil system, the initial state would depend on the state of the structure as well as on the state of the soil, such as degree of consolidation, water content etc. A change in the system stiffness may be due to changes only in the structure, only in the soil, or in both. While the changes due to damage within the structure are permanent, the soil may recover its original stiffness following strong shaking, but this may take time. Short-term temporal and amplitude variations in the system period can be studied by moving-window Fourier analysis [Udwadia and Trifunac, 1974] or zero-crossing analyses of individual earthquake recordings, and long term variations can be analyzed by comparison of results for different earthquakes. Trifunac et al. [2001b] performed such an analysis for the “rocking” response of the VN7SH building using 8 s windows for the 1971 San Fernando and 1994 Northridge earthquake data (Δt=0.02 s) and 4 s windows for the other earthquakes’ data (Δt=0.01 s). (They “extracted” the rocking response by band-pass filtering the total response between 0.1–0.2 Hz and 0.8–1.0 Hz using Ormsby filters.) Their results for individual records show progressive reduction of the system frequency, proportional to the intensity of motion. The comparison of results for different earthquakes indicates that the system frequency returned near its original value after most of the earthquakes. Based on the limited data for this building, “healing” of the soil seems to occur, probably resulting from dynamic consolidation caused by the aftershocks and by small earthquakes. Trifunac et al. [2001b] concluded that the soil-structure interaction affected significantly the building-soil response, and that the soil nonlinearity accounted for much of the reduction of the system stiffness. They considered such an effect to be beneficial for the building, because of dissipation of energy in the soil via nonlinear deformations, and consequent reduction of the energy exciting the structure. Finally, they speculated that, if founded on stiffer soil, this building would have suffered more severe damage from the
1994 Northridge earthquake. Besides its importance for explaining correctly the changes in the system stiffness, considering soil-structure interaction is also essential for correct estimation of the inter-story drift from earthquake recordings. For the 1994 Northridge earthquake, the drift angles of relative building deformation exceeded 0.004 and severe damage occurred. This drift was up to four times smaller than the apparent “drift” (computed based on the assumption that there is no soil-structure interaction).

1.3.4 Building Response to Ambient Noise

Ivanović et al. [1999] describe two detailed ambient vibration tests of the VN7SH building. The first one was carried out on February 4–5, 1994, about two weeks following the Northridge earthquake, and the second one was carried out two months later, on April 19–20, 1994, about a month after the M=5.3 aftershock of March 10, 1994. Total of six transducers were used, four Ranger SS-1 seismometers and two “old” Earth Sciences Rangers [Ivanović and Trifunac, 1995]. Ambient noise was recorded along one of the interior longitudinal frames (C), on each floor, at each of the nine columns, and for all three components of motion (L-longitudinal, T-transverse and V-vertical). The three Ranger SS-1 seismometers were used to record at all the measuring points. The values of the apparent frequencies for the first four modes of vibration obtained from the first experiment were: \( f = 1.0, 3.5, 5.7 \) and 8.1 Hz for the longitudinal (E-W) direction, and \( f = 1.4, 1.6, 3.9 \) and 4.9 Hz for the transverse (N-S) direction. The corresponding values obtained from the second experiment were: \( f = 1.1, 3.7, 5.7 \) and 8.5 Hz for the longitudinal (E-W) direction, and \( f = 1.4, 1.6, 3.9 \) and 4.9 Hz for the transverse (N-S) direction.

1.3.5 Ground Response to Ambient Noise

During the second ambient vibration experiment, detailed ambient noise measurements were conducted in the parking lot surrounding the building. Trifunac et al. [1999] presented their results in form of contours of amplitude and time delay of the ground motion relative to reference points on the ground floor of the building. The aim was to detect ground deformation associated with at least the fundamental transverse (1.4 Hz) and longitudinal (1.1 Hz) modes of vibration of the building [Foutch et al., 1975]. The overall pattern of the time delay, \( \tau \) (computed via cross-correlation), implies that the microtremor waves arrived from the west and scattered and diffracted around the building foundation. The measured delays for the horizontal components of motions imply apparent horizontal phase velocity of about 300 m/s, consistent with the interpretation that microtremors are high frequency Rayleigh waves propagating through the shallow soil layers. The delays for vertical motions also imply wave arrival from west and south-west, with apparent phase velocities 250 to 300 m/s.
2. DESCRIPTION OF THE BUILDING

The 7-story building described in this report (referred to as VN7SH) is located near the intersection of Roscoe Blvd. and San Diego Freeway (I-405) in the city of Van Nuys of metropolitan Los Angeles. Figure 2.1 shows a south-west view of the building. It was designed in 1965 [Blume et al., 1973], and constructed in 1966 at a cost of approximately $1.3 million. The total floor area is about 63,000 square feet. It served as a hotel in 1994 when it was severely damaged by the Northridge earthquake.

2.1 Structure

The structure is constructed of regular weight reinforced concrete. Table 2.1 gives the properties of the structural materials specified for the construction. The plan dimensions of the reinforced concrete structure are about 62×150 ft (Fig. 2.2) (1 ft=30.48 cm). The typical framing consists of columns spaced at 20 ft centers in the transverse direction and

<table>
<thead>
<tr>
<th>Table 2.1 Properties of the construction materials of the VN7SH building</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete (regular weight, 150 pcf (1))</strong></td>
</tr>
<tr>
<td>Location in the structure</td>
</tr>
<tr>
<td>Columns, 1st to 2nd floors</td>
</tr>
<tr>
<td>Columns, 2nd to 3rd floors</td>
</tr>
<tr>
<td>Beams and slabs, 2nd floor</td>
</tr>
<tr>
<td>All other concrete, 3rd floor to roof</td>
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</tbody>
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<tr>
<th><strong>Reinforcing steel</strong></th>
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<tr>
<td>Location in the structure</td>
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<tr>
<td>Beams and slabs</td>
</tr>
<tr>
<td>Column bars</td>
</tr>
</tbody>
</table>

(1) Pounds per cubic foot  
(2) Pounds per square inch  
(3) Kips per square inch
Fig. 2.1 VN7SH - South-West elevation
Fig. 2.2a  Typical floor framing plan

8 bays @ 18' - 9" = 150' - 0"
Fig. 2.2b Typical floor plan
Fig. 2.3 Typical transverse section
Fig. 2.4 Typical longitudinal section

<table>
<thead>
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</table>

8 bays @ 18' - 9" = 150' - 0"
Fig. 2.5 Log of typical soil boring

LEGEND:
A  Field moisture expressed as a percentage of the dry weight soil.
B  Dry density expressed in pounds per cubic foot.
C  Blows per foot of penetration using a 2000 pound hammer dropping 12'.
   * - a 1200 pound hammer was used.
   ■ Depth at which undisturbed sample was extracted.
Fig. 2.6 Foundation plan
19 ft centers in the longitudinal direction (Fig. 2.3 and 2.4). Spandrel beams surround the perimeter of the structure. Lateral forces in each direction are resisted by the interior column-slab frames and exterior column spandrel beam frames. The stiffness, in the exterior frames added by the spandrel beams, creates exterior frames that are roughly twice as stiff as the interior frames. With the exception of some light framing members supporting the stairway and elevator openings, the structure is essentially symmetric. The participation of the nonstructural brick filler walls and some exterior cement plaster in dynamic response could cause some asymmetry for lateral motion in the longitudinal direction [Islam, 1996].

The first floor is a slab on grade over about 2 ft of compacted fill. Except for two small areas at the ground floor covered by one-story canopies, the plan configurations of each floor, including the roof, are the same. The floor system is reinforced concrete flat slab, 10 inches thick at the second floor, 8.5 inches thick at the third to seventh floors and 8 inches thick at the roof (1 inch = 2.54 cm). A penthouse with mechanical equipment covers approximately 10 percent of the roof area.

The interior partitions are, in general, gypsum wallboard on metal studs. Cement plaster, one inch thick, is used for exterior facing at each end of the building and at the stair and elevator bays on the long side of the building. Double 16 gauge metal studs support the cement plaster. Some additional cement plaster walls are located on the south side of the building at the first floor. The north side of the building, along column line D (Fig. 2.3), has four bays of brick masonry walls located between the ground and the second floor at the east end of the structure. Nominal 1-inch expansion joints separate the walls from the underside of the second floor spandrels. Although none of the wall elements described are designed as a part of the lateral force-resisting system, they do contribute (to various degrees) to the stiffness of the structure.

2.2 Foundation System and Site Geology

The foundation system (Fig.2.6) consists of 38 inch deep pile caps, supported by groups of two to four poured-in-place 24-inch diameter reinforced-concrete friction piles. These are centered under the main building columns. All the pile caps are connected by a grid of the beams. Each pile is roughly 40 ft long and has design capacity of over 100 kips vertical load and up to 20 kips lateral load (1 kip=0.46 kg × 9.81 m/s² = 4.51 N).

The geological data indicate that the site lies on recent alluvium. A typical soil boring log, shown in Fig. 2.5, indicates that the underlying soil consists primarily of fine sandy silts and silty fine sands.
3. STRONG MOTION RECORDS

The first strong motion records in the VN7SH building, available so far in digitized form, are those of the 1971 San Fernando earthquake, and the largest recorded motions are those of the 1994 Northridge earthquake. Figure 3.1 shows the location of the building relative to the faults of the 1971 San Fernando and 1994 Northridge earthquakes. It also shows the epicenters of two Northridge aftershocks, and the direction of wave arrival for other earthquakes for which the records in this building are known to date and are cited in this report. All of these events are listed in Table 3.1, in chronological order. The earthquake magnitude, $M$, and epicentral distance, $R$, are also listed. The recorded peak accelerations are listed in Table 3.2.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake</th>
<th>Date</th>
<th>$M$</th>
<th>$R$ [km]</th>
</tr>
</thead>
<tbody>
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<td>San Fernando</td>
<td>02/09/1971</td>
<td>6.6</td>
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<tr>
<td>2</td>
<td>Whittier Narrows</td>
<td>10/01/1987</td>
<td>5.9</td>
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<td>3</td>
<td>Whitter-Narrows aft.</td>
<td>10/04/1987</td>
<td>5.3</td>
<td>38</td>
</tr>
<tr>
<td>4</td>
<td>Pasadena</td>
<td>10/03/1988</td>
<td>4.9</td>
<td>32</td>
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<td>Montebello</td>
<td>06/12/1989</td>
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<td>01/19/1989</td>
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<tr>
<td>7</td>
<td>Sierra Madre</td>
<td>06/28/1991</td>
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<td>8</td>
<td>Landers</td>
<td>06/28/1992</td>
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<td>9</td>
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<td>06/28/1992</td>
<td>6.5</td>
<td>149</td>
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<td>10</td>
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<td>01/17/1994</td>
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<td>Northridge aft.</td>
<td>12/06/1994</td>
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<td>10.8</td>
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</table>

The building response to the 1971 San Fernando earthquake was recorded by three self-contained triaxial AR-240 accelerographs, located in the building as shown in Fig. 3.2. These accelerograms were digitized manually, at a sampling rate > 50 samples per second [Trifunac and Lee 1973, Hudson et al., 1971; Trifunac et al., 1973]. From the simplified representation of the rupture history shown in Fig. 3.1 (simplified from Trifunac [1974]), it is seen that the first strong motion waves arrived from N 22° E and depth ~9 to 13 km. The rupture propagated with velocity of about 2 km/s up and towards south, and the last direct waves arrived 9 to 10 s later, from N 62° E. Figures 3.3a–c show the corrected accelerations, velocity and displacement at all the nine channels.
Fig. 3.1 Central San Fernando Valley and the site of VN7SH. The dashed lines represent the horizontal projections of the faults of the 1971 San Fernando and 1994 Northridge earthquakes. The duration of faulting during these two earthquakes was respectively 9 and 6 s. Directions and distances to seven other earthquakes are shown by arrows. The epicenters of two Northridge aftershocks are shown by solid stars.
Table 3.2  Peaks of instrument and baseline-corrected strong motion acceleration, velocity and displacement recorded in the VN7SH building between 1971 and 1994. The location and orientation of recording channels (1 through 9 for San Fernando earthquake, and 1 through 16 for all subsequent events) is shown in Figs 3.2 and 3.4.

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<td>Dis cm</td>
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Fig. 3.2 Location of the three AR-240 accelerographs which recorded 1971 San Fernando earthquake.
All the other earthquakes were recorded by a 13 channel CR-1 central recording system and one tri-component SMA-1 accelerograph with independent recording system, but common trigger time with the CR-1 recorder. The locations of all the 16 channels is shown in Fig. 3.4. On 1 October, 1987, the Whittier-Narrows earthquake occurred, and the recorded motions are shown in Fig. 3.5a–c. Before the 1992 Landers earthquakes, one Whittier-Narrows aftershock and four smaller close earthquakes were recorded. The recorded amplitudes of motion were small (see Table 3.2) and the time histories of processed data are not shown in this report. These records were used by Trifunac et al. [2001a,b] in their study of time and amplitude dependent changes of the system period. Two other “smaller” earthquakes occurred in the Los Angeles area, the $M=4.1$ Montebello earthquake of 12 June, 1989, and the $M=5.2$ Upland earthquake of 28 February, 1990, but did not trigger the recording system [Graizer, 1997]. The processed data of the larger, but more distant, 1992 Landers ($M=7.5$) and Big Bear ($M=6.5$) earthquakes are shown in Figs 3.6 and 3.7. The 1994 Northridge earthquake data are shown in Fig. 3.8 [Shakal et al., 1994]. Data from two Northridge aftershocks were used in our analyses, the $M=5.2$ aftershock of 20 March, 1994, and the $M=4.5$ aftershock of 6 December, 1994.

The instrumentation in the VN7SH building is operated by the Strong Motion Instrumentation Program of the California Division of Mines and Geology (CDMG). The records of events No. 2, 8, 9, 10 and 11 in Table 3.1 were digitized and released by CDMG. The records of events No. 3, 4, 5, 6, 7 and 12 were digitized at USC from Xerox copies in published reports (events 3–7) or from copies provided by the CDMG (event No. 12 [Graizer, 1997]). The LeAuto software package for automatic digitization of accelerograms and the LeBatch package for further processing of digitized data (instrument correction of recorded acceleration, filtering and computation of velocity and displacements, and computation of Fourier and Response Spectra) was used for this purpose [Trifunac and Lee, 1979; Lee and Trifunac, 1990].

There were many other $M > 4$ aftershocks between the main event and December of 1994. Table 3.3 lists 20 aftershocks (all recorded by 7 or more USC stations in the area), which may have been recorded in this building. At the time of writing of this report, no information has been available from the CDMG on which of these aftershocks have been recorded in this building.
Van Nuys, 7-storey Hotel
San Fernando Earthquake, 9 Feb., 1971, 06:00 PST, M_L=6.6, H=9 km

Fig. 3.3a  The first 23 s of acceleration recorded during the 1971 San Fernando earthquake. The sensor locations in the building and component orientations for the nine channels are shown in Figure 3.2.
Fig. 3.3b The first 23 seconds of velocities computed from recorded acceleration.
Van Nuys, 7-storey Hotel
San Fernando Earthquake, 9 Feb., 1971, 06:00 PST, $M_L = 6.6$, $H = 9$ km

Fig. 3.3c The first 23 seconds of displacements computed from recorded acceleration.
Fig. 3.4 Location and orientation of the thirteen sensors of the CR-1 recording system, and of the three sensors of the SMA-1 accelerograph.
Fig. 3.5a  The first 23 s of accelerations recorded during the 1987 Whittier-Narrows earthquake. Only for this earthquake, the sensitivity vector for channel 13 was directed towards south.
Fig. 3.5b The first 23 s of velocities computed from recorded accelerations.
Van Nuys, 7-storey Hotel
Whittier-Narrows Earthquake, 1 Oct. 1987, 14:42 GMT, $M_L=5.9$, $H=14$ km

Fig. 3.5c  The first 23 s of displacements computed from recorded accelerations.
Fig. 3.6a The first 45 s of recorded accelerations during the 1992 Landers earthquake.
Van Nuys, 7-storey Hotel  

Landers Earthquake, 28 Jun. 1992, 11:57 GMT, $M_L=7.5$, $H=5$ km

Fig. 3.6b The first 45 s of velocities computed from recorded accelerations.
Fig. 3.6c The first 45 s of displacements computed from recorded accelerations.
Fig. 3.7a  The first 22 s of accelerations recorded during the 1992 Big Bear earthquake.
Fig. 3.7b The first 22 s of velocities computed from recorded accelerations.
Fig. 3.7c The first 22 s of displacements computed from recorded accelerations.
Fig. 3.8a The first 23 s of accelerations recorded during the 1994 Northridge earthquake.
Van Nuys, 7-storey Hotel
Northridge Earthquake, 17 Jan. 1994, 12:30 GMT, $M_L = 6.4$, $H = 18$ km

Fig. 3.8b The first 23 s of velocities computed from recorded accelerations.
Fig. 3.8c The first 23 s of displacements computed from recorded accelerations.
Table 3.3 A list of $M > 4$ Events between 17 January and 17 October 1994 that triggered accelerographs of the Los Angeles Array Strong Motion Network (USC)

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* This column shows the number of USC stations which triggered (see also Todorovska et al 1999).
4. DESCRIPTION OF DAMAGE

The VN7SH building was damaged by the 1971 San Fernando and by the 1994 Northridge earthquakes. What follows is a brief review of the damage from the 1971 San Fernando earthquake published elsewhere, and a detailed description of the damage from the 1994 Northridge earthquake sequence documented on 4 February 1994 and on 19 April 1994 (following one of the larger aftershocks, of 20 March, 1994, $M = 5.2$).

4.1 Damage from the 1971 San Fernando Earthquake

The structural damage caused by this earthquake was relatively minor. It consisted of cracks in the spandrel beam-to-column connection at the north-east corner of the building. The damage to the partitions and exterior plaster was considerable. Epoxy was used to repair the spalled concrete of the second floor beam-column joints on the north side and east end of the building. The cost of this repair was less than $2,000. The nonstructural damage was extensive, and about 80% of all repair cost was used to fix the drywall partitions, bathroom tiles and plumbing fixtures. The damage was most severe on the second and third floors, and was minimal at the sixth and seventh floors. Forty-five bathtubs and 12 water closets had to be replaced in more than half of the bathrooms [Blume and Assoc. 1973]. The cost of repair was about $143,000.

Between 1971 and 1994, the building was shaken many times, but suffered no visible damage. For the 1971 San Fernando earthquake, at the foundation level, the peak horizontal acceleration was 0.25g and the peak horizontal velocity was 27 cm/s. Between 1971 and 1994, the largest recorded peak acceleration (0.16g) was that of the 1987 Whittier-Narrows earthquake, and the largest recorded velocity was that of the 1992 Landers earthquake (11 cm/s; see Table 3.2). During the 17 January 1994, Northridge earthquake, the peak recorded ground acceleration was 0.44g, peak velocity was 51 cm/s (EW component), and the building suffered considerable damage. Simple empirical criteria suggest that serious damage begins to occur for peak ground velocities 20 cm/s [Trifunac and Todorovska, 1997a,b, 1998; 1999].

4.2 Damage from the 1994 Northridge Earthquake and Early Aftershocks

4.2.1 First Damage Survey (February 4, 1994)

The Northridge earthquake of January 17, 1994, significantly damaged the building. At the time of this survey, the building was closed. The photographs of damage from this survey are presented in Appendix I.

Significant structural damage occurred in the exterior longitudinal frames (south-A and north-D frames), designed to take most of the lateral loads in the EW direction. In frame A (Fig. 4.1 south side), wide shear creaks appeared in the columns A3, A4, A5, A7 and
A8, just below the contact with the spandrel beam of the fifth floor. At contacts A5F5 (frame A, column 5, spandrel beam of the fifth floor) and A7F5, the cracks were 5 to 10 cm wide on the surface of the columns, and at contact A8F5 they were more than 10 cm wide. At all of these locations, buckling of the longitudinal bars due to the large relative motions was evident. At contact A8F5, the large deformation of the longitudinal bars caused deformation also of the transverse reinforcement. No visible damage was noticed along column A6 of frame A, although both of the adjacent columns (A5 and A7) were severely damaged. Frame A was covered from the exterior at the first floor, possibly covering some minor damages. At contacts A9F2 and A9F5, minor cracks were noticed, less than 1 cm wide.

A summary of the damage of frame A is shown schematically in Fig. 4.1, and Fig. 4.2 shows the areas of frame A covered by the photographs of the damage. Each rectangle corresponds to an area shown in the photos, and identified by a code describing the location of the photographed structural elements. For example, in code IA7F5-a,b, “I” indicated first survey, “A7” indicates column 7 of frame A, “F5” indicates fifth floor, and “a,b” indicate that there are two views, a and b, of the same location.

In exterior frame D, the shear cracks were moderately wide (0.2–1 cm) on the surface of the columns. A schematic representation of the damage of frame D (south view) is shown in Fig. 4.3, and Fig. 4.4 shows a map of the photographs its damage. At locations D5F4, D7F4, D8F5, D87F4 and D8F3, these cracks had a clear “x” shape. Column D1 cracked vertically along the third floor by about 1 cm. These cracks extended also into the spandrel beams of the third and fourth floors, as vertical cracks <1 cm wide. The cracks of columns D2, D3 and D4 at the first floor appeared to have been caused by the “short column” effect due to the brick wall, built up to 1/3 of the height of the columns in the first four bays. These cracks were about 0.5–1 cm wide (Fig. 4.3). A diagonal crack (about 0.5 cm wide) occurred at contact D9F2. At locations D7F2 and D9F2S, small diagonal cracks, less than 0.5 cm wide, were noted in the nonstructural elements.

Cracks were also seen between the bricks (through mortar) at the upper corners of the brick walls of the four eastern bays of the first floor (Fig. 4.3). There was no visible damage to the interior longitudinal frames (B and C), although large furniture (especially on the first floor), wallpaper and carpets could have hidden existing cracks. No damage was observed in the reinforced concrete slabs. On the 5th and 6th floors, small cracks, less than 0.2 cm wide, were noticed in the slab around the central columns.

There were no signs of large deformations in the foundation. There were no cracks both at the first floor slab and in the pavement around the building.
Fig. 4.1 Frame A – a schematic representation of damage (first survey, Feb. 4, 1994).
Fig. 4.2 Frame A – schematic representation of views of photographs of damage (first survey, Feb. 4, 1994; see Appendix I).
Fig. 4.3 Frame D- schematic presentation of damage (first survey, Feb. 4, 1994).
Fig. 4.4  Frame D – schematic representation of views of photographs of damage (first survey, Feb. 4, 1994; see Appendix I).
In the east side of the building, minor (nonstructural) cracks were seen, mainly along the first floor. The widths of all cracks were determined from the photographs of damage.

The nonstructural damage of the building was extensive. Every guest-room experienced some nonstructural damage. On the upper floors (above 3rd), the furniture was overturned. The wallpaper was distorted or torn off, due to large relative motions and deformations of the interior walls. Relative displacements also caused extensive damage to the brittle ceramic tile covers in the bathrooms. There were numerous cracks in the bathtubs, and many ceramic tiles fell off.

4.2.2 Second Damage Survey (April 19-20, 1994)

Between the first and second damage surveys (when also ambient vibration experiments were carried out), the building was restrained to support the weakened structural elements. Wooden braces were placed at selected bays of the exterior longitudinal frames (at the 2nd through 4th floors of frame A, and at the 1st through fourth floors of frame D), and also at the first floor of the interior (B and C) frames. There were no braces along the transverse spans of the building. The location of the braces is shown in the Figs 4.5–4.9. For braces on the first floor of the exterior frames, ~18×18 cm wooden beams were used, and at the other floors 15×15 cm wooden beams were used. We do not know whether the braces were placed before or after the March 20 aftershock, which could have contributed additional damage. During this survey, large shear cracks, with deformations of the longitudinal reinforcement larger than those observed during the first survey, were noticed in the frame A. The cracks were wider at locations A3F5 (column A3, floor slab 5), A4F5 and A7F5. No significant differences were noticed at locations A5F5 and A8F5. Figure 4.10 shows schematically a summary of the changes in the damage of frame A. No additional damage was noticed in the north (D) frame or in the interior frames. Also, we did not notice any significant increase in nonstructural damage.

The photographs of damage are shown in Appendix II. Figures 4.11 and 4.12 show the locations of the photographed structural elements in frames A and D. The codes for the figure captions (“addresses”) are the same as those in Appendix I. (For example IIA7F5a-f designates II-second experiment, column A7, floor five slab, and that there are six (a,b,c,d,e,f) different views of the same element).
Fig. 4.5 Frame A - location of the braces (second survey, April 19-20, 1994).
Fig. 4.6 Frame B - location of the braces (second survey, April 19-20, 1994).
Fig. 4.7 Frame C - location of the braces (second survey, April 19-20, 1994).
Fig. 4.8 Frame D - location of the braces (second survey, April 19-20, 1994).
Fig. 4.9 First floor - location of the braces (second survey, April 19-20, 1994).
Fig. 4.10 Frame A - changes in damage between the first (Feb. 4, 1994) and second (Apr. 19, 1994) surveys.
Fig. 4.11 Frame A - schematic representation of views of photographs of damage (second survey, April 19-20, 1994).
Fig. 4.12 Frame D - schematic representation of views of photographs of damage (second survey, April 19-20, 1994).
5. SOME DESIGN CONSIDERATIONS AND MAXIMUM RECORDED DRIFTS

The most severe damage of the VN7SH building from the 1994 Northridge earthquake occurred just below the 5th floor slab, in the south exterior frame (A). The following presents an approximate comparison of the design strength capacity of the exterior frame columns with the loads associated with the maximum recorded drifts.

The exterior columns (14×20 in) had their weak axis oriented along the transverse building direction (see Fig. 5 and Fig. 7 in Blume and Assoc., 1973). The cross section of the spandrel beams was 16×22 in. This results in higher stiffness and building frequency for the NS direction ($f \sim 1.4$ Hz) relative to the EW direction ($f \sim 1.0$ Hz). The interior columns contributed equally to the building stiffness in both directions, and had cross-section 20×20 in between the ground and second floors, and 18×18 in above the second floor.

5.1 Nominal Moment Capacity and Moments Evaluated from the Recorded Drift

Figure 5.1 shows a summary of the capacities and of the moments caused by the drifts from the Northridge earthquake, computed from the recorded response. The nominal moment capacities for the exterior columns (EW and NS directions) were calculated from the known geometry, area of reinforcement and for assumed location of the reinforcing bars (Fig. 5.1). The material characteristics used in these calculations follow from those in Table 2.1. Minimal compressive strength $f_c' = 4000$ psi was used for the columns between the 2nd and 3rd floors, and $f_c' = 3000$ psi for the columns above the 3rd floor ($f_c' = 4000$ psi was assumed for the columns between the 1st and 2nd floors). The respective moduli of elasticity used were $E = 3.7 \times 10^6$ psi and $E = 3.3 \times 10^6$ psi. For the reinforcing steel, minimal specified yield strength $f_y = 60$ ksi and modulus of elasticity $E = 29 \times 10^6$ psi were used. The interaction diagrams were computed for $\phi = 0.7$, and were taken from Leet (1984) as follows: in Fig. 7.37 for $\rho_g = 0.021$ and $\rho_g = 0.013$, in Fig. 7.41 for $\rho_g = 0.036$, and in Fig. 7.43 for $\rho_g = 0.021$ and $\rho_g = 0.013$. Nominal “balanced” moments and moments corresponding to “pure” bending (large vertical accelerations can result in zero axial forces in the columns) were considered. Those estimates are shown in Fig.5.1 by heavy and by light continuous lines respectively, and by the shaded area between them.

The moments caused by the drift from the Northridge earthquake were calculated for each story using the moduli of elasticity and cross-section moments of inertia for the transformed cross-sections. For clamped-clamped boundary conditions (the extreme case of high stiffness), the moments are shown in Fig. 5.1 by heavy dashed lines. To approximate the actual end stiffness of the columns, the effect of the neighboring columns and beams was approximated by using slope-deflection analysis for a two-story-
Fig. 5.1 Comparison of the nominal moment capacities of the columns (the continuous heavy line represents “balanced” point while the light continuous line corresponds to pure bending with zero axial force) with moments caused by the drift associated with relative building deformation (high-pass filtered data, heavy dashed lines) and with “moments” corresponding to the “rocking” of the whole building (low-pass filtered data, heavy dotted lines).
two-bay frame with spans equal to 6.0 m and story heights equal to 3.0 m. The moments caused by the story drifts $\Delta$ ($2\Delta$ on the “second floor” and $\Delta$ on the “first floor”) were next calculated. It was assumed that the floor slabs participate in the beam stiffness. In the calculation of the beam stiffness, lengths equal to $3d$ were added to one side of the beams, where $d=$slab thickness=8.5 inches. This approximate analysis gave moment reduction factor $f=2.7$, and related the moments at the middle column of a two story frame to the moments of the clamped-clamped column caused by the same horizontal displacement $\Delta$. A reduction factor of 3.6 was used to approximate the moments in the transverse direction.

The heavy dashed lines in Fig. 5.1 show the moments associated with the peak drifts during the Northridge earthquake, measured from high-pass filtered data. At the east side of the building, these drifts exceeded $0.003H$ between the 6th floor and the roof. Table 5.1 shows the peak EW drifts for both low-pass and high-pass filtered data for the 1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear and 1994 Northridge earthquakes. For all earthquakes and both types of filtered data, the drifts at the roof relative to the 6th floor are smaller than the drifts at the lower floors. For the Northridge earthquake, the drifts computed from the high-pass filtered data were the largest at the middle floors. Those drifts were probably responsible for the damage of the columns below the 5th floor.

Table 5.1 Peak longitudinal (EW) drift angles.

<table>
<thead>
<tr>
<th>Earthquake name</th>
<th>Date</th>
<th>Drift angles computed from low-pass filtered data</th>
<th>Drift angles computed from high-pass filtered data</th>
<th>Drift angles computed from broad-band data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whittier-Narrows</td>
<td>10/01/1987</td>
<td>0.00060</td>
<td>0.00150</td>
<td>0.00160</td>
</tr>
<tr>
<td>Landers</td>
<td>06/28/1992</td>
<td>0.00080</td>
<td>0.00130</td>
<td>0.00180</td>
</tr>
<tr>
<td>Big Bear</td>
<td>06/28/1992</td>
<td>0.00030</td>
<td>0.00080</td>
<td>0.00180</td>
</tr>
<tr>
<td>Northridge</td>
<td>01/17/1994</td>
<td>0.00438</td>
<td>0.01330</td>
<td>0.01450</td>
</tr>
</tbody>
</table>
Table 5.2  Peak transverse (NS) drift angles at the East and West sides of the building.

<table>
<thead>
<tr>
<th>Earthquake name</th>
<th>Date</th>
<th>East side</th>
<th>West side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><a href="#">Drift angles computed from low-pass filtered data</a></td>
<td><a href="#">Drift angles computed from high-pass filtered data</a></td>
</tr>
<tr>
<td>Whittier-Narrows</td>
<td>10/01/1987</td>
<td>0.00064 0.00050 0.00084 0.00016</td>
<td>0.00081 0.00124 0.00124 0.00121</td>
</tr>
<tr>
<td>Landers</td>
<td>06/28/1992</td>
<td>0.00118 0.00149 0.00206 0.00220</td>
<td>0.00294 0.00484 0.00500</td>
</tr>
<tr>
<td>Big Bear</td>
<td>06/28/1992</td>
<td>0.00069 0.00051 0.00104 0.00114</td>
<td>0.00064 0.00100 0.00124</td>
</tr>
<tr>
<td>Northridge</td>
<td>01/17/1994</td>
<td>0.00347 0.01039 0.00983 0.00988</td>
<td>0.00247 0.00267 0.00227 0.00434</td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="#">Drift angles computed from high-pass filtered data</a></td>
<td><a href="#">Drift angles computed from broad-band data</a></td>
</tr>
<tr>
<td>Whittier-Narrows</td>
<td>10/01/1987</td>
<td>0.00059 0.00046 0.00056 0.00089</td>
<td>0.00397 0.00980 0.01154 0.01097</td>
</tr>
<tr>
<td>Landers</td>
<td>06/28/1992</td>
<td>0.00014 0.00035 0.00048 0.00045</td>
<td>0.00298 0.00841 0.01040 0.00759</td>
</tr>
<tr>
<td>Big Bear</td>
<td>06/28/1992</td>
<td>0.00024 0.00030 0.00038 0.00033</td>
<td>0.00064 0.00100 0.00124</td>
</tr>
<tr>
<td>Northridge</td>
<td>01/17/1994</td>
<td>0.00247 0.00267 0.00227 0.00434</td>
<td>0.00247 0.00267 0.00227 0.00434</td>
</tr>
</tbody>
</table>
Table 5.2 shows the NS drifts, computed for the east and west sides of the building. The heavy dotted lines in Fig.5.1 show the moments associated with the peak drifts measured from the low-pass filtered data. For the NS motion, these drifts approximate the rigid-body rocking response of the soil-structure system. These “drift” amplitudes (between the 2nd floor and the roof) were more than twice the drifts allowed by the code. The data was filtered by Ormsby filters with frequencies 0.8–1.0 to 23–35 Hz for the high-pass filtered data. For the low-pass filtered data, the role-off and cut-off frequencies varied between 0.8–1.0 and 0.95–1.15 Hz depending on the amplitudes of motion.

5.2 Comparison of the Moments

Figure 5.1 shows that, during the Northridge earthquake and for the transverse direction, the high-pass filtered response exceeded the nominal moment capacities above the 4th floor (assuming clamped-clamped boundary conditions). For moment reduction factor equal to 2.7, it resulted in comparable but smaller than the nominal moments.

Combining the overall results for the longitudinal and transverse responses, it is evident that, for the high-pass filtered drifts, the moment capacity of the exterior columns between the 4th and 6th floors was exceeded. Even though such an analysis is very rough and approximate, these results agree with the observed damage in the vicinity of the 5th floor slab.

The moments computed for the “drifts” of the soil-structure system are much larger than the moment capacities for both NS and EW directions. These large rocking displacements (particularly for the NS response) result from soil deformation and approximately rigid body rocking, and thus do not contribute directly to the moments associated with the relative (floor to floor) drift displacements.
6. REFERENCES


APPENDIX I

Appendix I presents all photographs taken at the time of the first damage survey of the VN7SH building, which took place on 4 April, 1994. The location of the photographed elements is identified in Figures 4.2 and 4.4.
Figure A.I.1  Photograph A-a.
Figure A.I.2  Photograph A-b.
Figure A.I.3  Photograph A-c.
Figure A.I.4  Photograph A-d.
Figure A.I.6   Photograph A4-5F4-5.
Figure A.I.7  Photograph A3F5-a.
Figure A.I.8  Photograph A3F5-b.
Figure A.I.9  Photograph A3F5-c.
Figure A.I.10 Photograph A4F5-a.
Figure 11.11 Photograph A4F5-b.
Figure A.I.12  Photograph A4F5-c.
Figure A.I.13  Photograph A5F5-a.
Figure A.I.14  Photograph A5F5-b.
Figure A.I.15    Photograph A5F5-c.
Figure A.I.16  Photograph A5-F5-d.
Figure A.I.17  Photograph A5F5-e.
Figure A.I.18  Photograph A7F5-a.
Figure A.I.19  Photograph A7F5-b.
Figure A.I.20  Photograph A8F5-a.
Figure A.I.21 Photograph A8F5-b.
Figure A.I.22  Photograph A9F5.
Figure A.I.23 Photograph D-a.
Figure A.I.24   Photograph D-b.
Figure A.I.25  Photograph D-c.
Figure A.I.26  Photograph D-d.
Figure A.I.27  Photograph D1F3-4-a.
Figure A.I.28  Photograph D1F3-4-b.
Figure A.I.29  Photograph D1F3.
Figure A.I.30  Photograph D4F4.
Figure A.I.31   Photograph D4F1-a.
Figure A.I.32 Photograph D4F1-b.
Figure A.I.33  Photograph D7F4.
Figure A.I.34  Photograph D7F5.
Figure A.I.35 Photograph D8F3.
Figure A.I.36  Photograph D9F2.
APPENDIX II

Appendix II presents all photographs taken at the time of the second damage survey of the VN7SH building, which took place on 19 April, 1994. The location of the photographed elements is identified in Figures 4.11 and 4.12.
Figure A.II.1 Photograph A-a.
Figure A.II.2  Photograph A-b.
Figure A.II.3 Photograph A-c.
Figure A.II.4 Photograph A-d.
Figure A.II.5  Photograph A-e.
Figure A.II.6 Photograph A3F5-a.
Figure A.II.7  Photograph A3F5-b.
Figure A.II.8  Photograph A4F5-a.
Figure A.II.9 Photograph A4F5-b.
Figure A.II.10    Photograph A5F5-a.
Figure A.II.11 Photograph A5F5-b.
Figure A.II.12    Photograph A5F5-c.
Figure A.II.13  Photograph A5F5-d.
Figure A.II.14  Photograph A7F5-a.
Figure A.II.15  Photograph A7F5-b.
Figure A.II.16     Photograph A7F5-c.
Figure A.II.17  Photograph A7F5-d.
Figure A.II.18  Photograph A7F5-e.
Figure A.II.19  Photograph A7F5-f.
Figure A.II.20    Photograph A8F5-a.

A.II–21
Figure A.II.21 Photograph A8F5-b.
Figure A.II.22  Photograph A8F5-c.
Figure A.II.23  Photograph D-a.
Figure A.II.24  Photograph D-b
Figure A.II.25   Photograph D-c.
Figure A.II.26      Photograph D2F1.
Figure A.II.27 Photograph D3F1.
Figure A.II.28 Photograph D4F1.
Figure A.II.29  Photograph D6F4-5.
Figure A.II.30  Photograph D7F4-5.