EVALUATION OF CURRENT NONLINEAR STATIC PROCEDURES FOR CONCRETE BUILDINGS USING RECORDED STRONG-MOTION DATA

By

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This study evaluates current Nonlinear Static Procedures (NSPs) specified in the FEMA-356, ASCE-41, ATC-40, and FEMA-440 documents using strong-motion data from reinforced-concrete buildings. For this purpose, three-dimensional computer models of five reinforced concrete buildings – Imperial County Services Building, Sherman Oaks Commercial Building, North Hollywood Hotel, Watsonville Commercial Building, and Santa Barbara Office Building – are developed. When appropriate, springs at the building’s base are included to account for the soil-structure interaction effects. These buildings are selected because they were strongly shaken, several deformed beyond their linear-elastic range, during past earthquakes and their recorded motions are available. The recorded motions are interpolated to obtain motions at non-instrumented floors. These motions are used to derive seismic demands – peak roof (or target node) displacement, floor displacements, story drifts, story shears, and story overturning moments. The pushover curves are developed from nonlinear static analysis of computer models of these buildings and various demands estimated from the NSP methods.

A comparison of peak roof (or target node) displacements estimated from the NSPs with the value derived from recorded motions shows that: (1) the NSPs either overestimate or underestimate the peak roof displacement for several of the buildings considered in this investigation; (2) the ASCE-41 Coefficient Method (CM), which is based on recent improvements to the FEMA-356 CM suggested in FEMA-440 document, does not necessarily provide a better estimate of roof displacement; and (3) the improved FEMA-440 Capacity Spectrum Method (CSM) generally provides better estimates of peak roof displacements compared to the ATC-40 CSM. However, there is no conclusive evidence that either the CM procedures (FEMA-356 or ASCE-41) or the CSM procedure (ATC-40 or FEMA-440) lead to a
better estimate of the peak roof displacement when compared with the value derived from recorded motions.

A comparison of the height-wise distribution of floor displacements, story drifts, story shears and story overturning moments indicates that the NSP provides: (1) reasonable estimate of floor displacements; (2) poor estimate of drifts in upper stories due to its inability to account for higher mode effects; (3) very poor, and possibly unreliable, estimates of story shears and story overturning moments.

A comparison of pushover curves from various computer programs using different modeling assumptions led to significantly different pushover curves. This indicates significant sensitivity of pushover curves to modeling assumptions which may potentially lead to different results and conclusions from the NSP.
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CHAPTER 1. INTRODUCTION

Estimating seismic demands at various performance levels, such as life safety and collapse prevention, requires explicit consideration of inelastic behavior of the structure. While nonlinear response history analysis (RHA) is the most rigorous procedure to compute seismic demands, current structural engineering practice prefers to use the nonlinear static procedure (NSP) or pushover analysis. The two key steps in estimating seismic demands in NSP are: (1) estimation of the target node displacement; and (2) pushover analysis of the structure subjected to monotonically increasing lateral forces with specified height-wise distribution until the target displacement is reached. Both the force distribution and target displacement are typically based on the assumption that the response is controlled by the fundamental mode and that the mode shape remains unchanged after the structure yields.

The two widely used procedures to estimate the target displacement are: (1) the Coefficient Method (CM) defined in the FEMA-356 document (ASCE, 2000); and (2) the Capacity Spectrum Method (CSM) specified in ATC-40 document (ATC-40, 1997). The CM utilizes a displacement modification procedure in which several empirically derived factors are used to modify the response of a linearly-elastic, single-degree-of-freedom (SDOF) model of the structure. The CSM is a form of equivalent linearization. This technique uses empirically derived relationships for the effective period and damping as a function of ductility to estimate the response of an equivalent linear SDOF oscillator.

Various researchers have found that the CM and CSM may provide substantially different estimates of target displacement for the same ground motion and the same building (Aschheim et al., 1998; Akkar and Metin, 2007; Chopra and Goel, 2000; Goel, 2007; Miranda and Ruiz-Garcia, 2002) and have proposed improved procedures for estimating the target displacement. The ATC-55 project, which led to publication of the FEMA-440 document (ATC-55, 2003),
undertook a comprehensive examination of the existing research in this area and has proposed improvements to both the CM and CSM.

Most previous investigations on development and evaluation of NSPs are based on numerical modeling studies; a comprehensive list of previous investigations is available in the FEMA-440 document (ATC-55, 2003). Recorded motions of strongly shaken buildings, especially those deformed into the inelastic range, provide a unique opportunity to evaluate such procedures. Therefore, the principal objective of this investigation is to evaluate the current NSPs for seismic analysis and evaluation of building structures using strong-motion records of reinforced-concrete buildings. The NSPs to be evaluated are: (1) Coefficient Method in the FEMA-356 document (ASCE, 2000); (2) Capacity Spectrum Method in the ATC-40 report (ATC-40, 1997); and (3) improved Coefficient Method in ASCE-41 document (ASCE, 2007); and (4) improved Capacity Spectrum Method proposed in the FEMA-440 document (ATC, 2005). The accuracy of the NSP is evaluated by comparing several seismic demands – peak roof displacement, floor displacements, story drifts, story shears, and story overturning moments – computed from various the NSP with those derived directly from recorded motions.

This report is organized as follows: Chapter 2 describes the selected buildings and their recorded motions used in this investigation; Chapter 3 presents analysis of recorded data to estimate vibration periods and identify nonlinearity in response of selected building; Chapter 4 describes the procedure to interpolate motions at non-instrumented floors; Chapter 5 describes the analytical models used in this investigation along with mode-shapes and vibration periods of the selected buildings; Chapter 6 summarizes the current nonlinear static procedures; Chapter 7 examines the effects of various modeling assumptions on pushover curve; Chapter 8 presents pushover curves of the selected buildings; Chapter 9 examines the nonlinear static procedures; and Chapter 10 presents summary and conclusions of this study.
CHAPTER 2. SELECTED BUILDINGS AND STRONG-MOTION DATA

Recorded motions of buildings that were strongly shaken and potentially deformed beyond the yield limit during the earthquake are required for this investigation. For this purpose, five concrete buildings, ranging from low-rise to high-rise, have been selected (Table 1). The strong-motion data used in this investigation are identified in Table 1 for each building. The recorded strong-motion data for each of the selected buildings is available from the US National Center for Engineering Strong Motion Data (NCESMD) (http://www.strongmotioncenter.org). Following is a brief description of each of the five selected buildings and the recorded strong-motion data.

<table>
<thead>
<tr>
<th>Buildings name</th>
<th>CSMIP Station</th>
<th>Number of Stories</th>
<th>Strong-Motion Data from</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial County Services Building, El Centro</td>
<td>01260</td>
<td>6/0</td>
<td>1979 Imperial Valley Earthquake</td>
</tr>
<tr>
<td>13-Story Commercial Building, Sherman Oaks</td>
<td>24322</td>
<td>13/2</td>
<td>1994 Northridge Earthquake</td>
</tr>
<tr>
<td>20-Story Hotel, North Hollywood</td>
<td>24464</td>
<td>20/1</td>
<td>1994 Northridge Earthquake</td>
</tr>
<tr>
<td>4-Story Commercial Building, Watsonville</td>
<td>47459</td>
<td>4/0</td>
<td>1989 Loma Prieta Earthquake</td>
</tr>
<tr>
<td>3-Story UCSB Office Building, Santa Barbara</td>
<td>25213</td>
<td>3/0</td>
<td>1978 Santa Barbara Earthquake</td>
</tr>
</tbody>
</table>

**Imperial County Services Building**

The Imperial County Services Building has open first story and five occupied stories (Figure 2.1). Designed in 1968, its vertical load carrying system consists of 12.7 cm (5 inch) reinforced-concrete (RC) thick slabs supported by RC pan joists spanning in transverse direction, which in turn are supported by RC frame spanning in the longitudinal direction. The lateral load
system consists of RC shear walls in the transverse direction and moment resisting frames in the longitudinal direction. The shear walls are offset in the first story compared to upper stories. The foundation system consists of piles under each column with pile caps connected with RC beams.

Figure 2.1. Imperial County Services Building (http://www.strongmotioncenter.org).

The Imperial County Services building was severely damaged during the 1979 Imperial County earthquake. The primary damage included failure of the 1st story columns at the base on the east side of the building (Figure 2.2a). This failure occurred on a column cross section with low confining steel (Figure 2.2b). This building was found to be so badly damaged during the 1979 earthquake that it was subsequently demolished. A comprehensive report on damage of this building is available elsewhere (ATC-9, 1984).

The Imperial County Services building was instrumented in 1976 with 13 sensors at four levels of the building and 3 sensors at a reference free-field site. The sensors in the building measure horizontal accelerations at ground floor, 2nd floor, 4th floor, and roof; and vertical
acceleration at ground floor (Figure 2.3). The free-field site (CSMIP Station No. 01335) is located about 103.6 m (340 ft) east and 3.05 m (10 ft) north of the north-east corner of the building.

Figure 2.2. Damage to the Imperial County Services building during the 1979 Imperial County earthquake: (a) First-story columns on the east-side of the building; and (b) Close-up view of the column base (http://nisee.berkeley.edu/bertero/html/slides.html).

Figure 2.3. Sensor location in the Imperial County Services Building.

The recorded motions of this building are available for the 1979 Imperial Valley earthquake. The corrected accelerations obtained from accelerations recorded within the
structure during the 1979 earthquake are presented in Figure 2.4. The peak recorded accelerations at the roof are 0.581g at channel 3 in the transverse (north-south) direction and 0.453g at channel 4 in the longitudinal (east-west) direction. The peak accelerations at the base of the building are 0.337g at channel 10 in the transverse (north-south) direction and 0.332g at channel 13 in the longitudinal (east-west) direction. There is an unusual spike in acceleration (0.655g) at channel 9 located on the east edge of the 2nd floor the building; no such spike is visible in acceleration histories of two other channels located on this floor (channels 7 and 8) where the peak accelerations are 0.363g and 0.314g. Recall from Figure 2.2 that 1st story columns on the east side – the same side of the building as channel 9 – failed during the earthquake. It is believed that this spike in acceleration at channel 9 may be due to progressive collapse on this side of the building.

The corrected accelerations at the free-field site are presented in Figure 2.5 with orientation of channel 1 in the east-west direction, channel 2 in the vertical direction, and channel 3 in the north-south direction. Note that the north-axis of the free-field sensors is not exactly aligned with that of the building sensors but rotated clockwise by an angle of approximately 2-degrees. The peak free-field accelerations of 0.213g in the north-south direction and 0.236g in the east-west direction (Figure 2.5) are slightly lower than 0.337g and 0.332g recorded at the base of the building in the respective directions (Figure 2.4). However, the peak free-field vertical acceleration of 0.236g (Figure 2.5) is higher than 0.178g recorded by channel 12 at base of the building (Figure 2.4). Obviously, these differences suggest soil-structure interaction effects.
Figure 2.4. Accelerations recorded at the Imperial County Services Building during the 1979 Imperial County earthquake.
Figure 2.5. Accelerations recorded at free-field site of the Imperial County Services Building during the 1979 Imperial County earthquake.

**Sherman Oaks Commercial Building**

The Sherman Oaks Commercial Building has 13 stories above and two floors below the ground (Figure 2.6). Designed in 1964, its vertical load carrying system consists of 12.4 cm (4.5 inch) thick slabs supported by concrete beams, girders, and columns. The lateral load system consists of moment resisting concrete frames in the upper stories and concrete shear walls in the basements. The foundation system consists of concrete piles.

The Sherman Oaks building, often referred to as a non-ductile reinforced-concrete building, was damaged during the 1994 Northridge earthquake. The damage included cracks at many beam-column joints during the 1994 Northridge earthquake (Shakal et al., 1994). This building was strengthened with friction dampers after the Northridge earthquake.

The Sherman Oaks building was instrumented in 1977 with 15 sensors on five levels of the building. The sensors in the building measure horizontal accelerations at the 2nd sub-basement level, ground level, 2nd floor, 8th floor, and roof level (Figure 2.7). Vertical accelerations were measured at the 2nd sub-basement (Figure 2.7). No free-field site was instrumented for this building.

Although the Sherman Oaks building yielded recorded motions during four major
earthquakes – 1994 Northridge, 1992 Landers, 1991 Sierra Madre, and 1987 Whittier – the strongest shaking occurred during the 1994 Northridge earthquake. The corrected accelerations from the Northridge earthquake are used in this investigation and are presented in Figure 2.8. The peak recorded accelerations at the roof are 0.618g at channel 3 in the transverse (north-south) direction and 0.257g at channel 1 in the longitudinal (east-west) direction. The peak accelerations at the base (2nd sub-level) of the building are 0.446g at channel 15 in the transverse (north-south) direction and 0.214g at channel 13 in the longitudinal (east-west) direction.

Figure 2.6. Sherman Oaks Commercial Building.

The building is instrumented with three sensors per floor: two in the transverse direction and one in the longitudinal direction. The accelerations at locations of the two transverse sensors can be used to detect presence of torsional motions (or asymmetry) in the building: the accelerations at the two sensors should essentially be the same in symmetric building whereas
differential accelerations are expected in asymmetric buildings. An examination of two sensors in the transverse direction on the same floor indicates presence of torsional motions in the Sherman Oaks building. For example, peak accelerations recorded by channel 3 located on west end of the roof is 0.618g which is significantly higher than the peak acceleration of 0.464g recorded by channel 2 located at the center of the roof. Similar differential accelerations also occur at other floors.

![Diagram of sensor locations in the Sherman Oaks Commercial Building](image)

Figure 2.7. Sensor location in the Sherman Oaks Commercial Building.

The sensors at ground level of the building recorded very high accelerations – 0.866g in the transverse direction at channel 11 and 0.373g in the longitudinal direction at channel 10 – compared to any other location of the building. An examination of the structural plans indicates significant stiffness discontinuity in the building at the ground level: a very stiff system in bottom two stories (2nd Sub-Level to Ground Floor) with a combination of moment-resisting frames, stiff shear walls, and soil surrounding, which transitions at the ground level to a moment-resisting frame in upper stories.
Figure 2.8. Accelerations recorded at the Sherman Oaks building during the 1994 Northridge earthquake.
North Hollywood Hotel

The North Hollywood Hotel has 20 stories above and one floor below the ground (Figure 2.9). Designed in 1966, its vertical load carrying system consists of 12.4 cm (4.5 inch) to 15 cm (6 inch) thick RC slabs supported by concrete beams and columns. The lateral load system consists of ductile moment resisting concrete frames in both directions. The foundation system consists of spread footing below columns.

Figure 2.9. North Hollywood Hotel.

This building was instrumented in 1983 with 16 sensors on five levels of the building. The sensors in the building measure horizontal accelerations at the basement level, 3rd floor, 9th floor, 16th floor, and roof level; and vertical acceleration at the basement (Figure 2.10). No free-field site was instrumented for this building.

Although the North Hollywood building yielded recorded motions during three major earthquakes – 1994 Northridge, 1991 Sierra Madre, and 1987 Whittier – the strongest shaking occurred during the 1994 Northridge earthquake. The corrected accelerations from the Northridge earthquake are used in this investigation and are presented in Figure 2.11. The peak recorded accelerations at the roof are 0.653g at channel 2 in the transverse (north-south)
direction and 0.312g at channel 10 in the longitudinal (east-west) direction. The peak accelerations at the base (Basement) of the building are 0.317g at channel 1 in the transverse (north-south) direction and 0.113g at channel 14 in the longitudinal (east-west) direction.

As was the case for the Sherman Oaks building, the North Hollywood building is also instrumented with three sensors per floor: two in the transverse direction and one in the longitudinal direction. An examination of two sensors in the transverse direction on the same floor indicates the presence of torsional motion in the North Hollywood building as well.

Figure 2.10. Sensor location in the North Hollywood Hotel.
Figure 2.11. Accelerations recorded at the North Hollywood building during the 1994 Northridge earthquake.
Watsonville Commercial Building

The Watsonville Commercial Building has 4 stories above the ground (Figure 2.12). Originally designed and constructed in 1948 as a three-story building, a fourth story was added in 1955. Its vertical load carrying system consists of concrete slabs supported by concrete-encased columns. The lateral load system consists of concrete shear walls in both directions. The foundation system consists of spread footing below shear walls.

Figure 2.12. Watsonville Commercial Building.

This building was instrumented in 1982 with 13 sensors on three levels of the building. The sensors in the building measure horizontal accelerations at the ground floor, 3rd floor, and roof level; and vertical accelerations at four corners of the building at the ground floor (Figure 2.13). No free-filed site is available for this building.

This building yielded motions during 1989 Loma Prieta earthquake. The corrected accelerations from this earthquake are presented in Figure 2.14. The peak recorded accelerations at the roof are 0.79g at channel 6 in the transverse (north-south) direction and 1.2g at channel 11.
in the longitudinal (east-west) direction. The peak horizontal accelerations at the base (Ground Floor) of the building are 0.27g at channel 9 in the transverse (north-south) direction and 0.36g at channel 13 in the longitudinal (east-west) direction. An examination of two sensors in the transverse direction on the same floor indicates presence of torsional motion in the Watsonville building as well.

The Watsonville building is also instrumented by four sensors (channels 1 to 4) at the base to measure vertical accelerations. The accelerations recorded by these sensors may be used to detect the presence of rocking in the building due to soil-flexibility. It appears that this building experienced rocking at the base in both the east-west and north-south directions: differential peak accelerations at channels 1 and 2 of 0.57g and 0.53g indicate rocking in the east-west direction and at channels 1 and 3 of 0.57g and 0.51g indicate rocking in the north-south direction.

Figure 2.13. Sensor location in the Watsonville Commercial Building.

The Watsonville building is also instrumented by four sensors (channels 1 to 4) at the base to measure vertical accelerations. The accelerations recorded by these sensors may be used to detect the presence of rocking in the building due to soil-flexibility. It appears that this building experienced rocking at the base in both the east-west and north-south directions: differential peak accelerations at channels 1 and 2 of 0.57g and 0.53g indicate rocking in the east-west direction and at channels 1 and 3 of 0.57g and 0.51g indicate rocking in the north-south direction.
Figure 2.14. Accelerations recorded at the Watsonville Commercial Building during the 1989 Loma Prieta earthquake.

**Santa Barbara Office Building**

This Santa Barbara Office Building on the campus of University of California at Santa Barbara has 3 stories above the ground (Figure 2.15). Originally designed and constructed in 1960, this building was strengthened in 1975 with shear walls in both directions. The vertical
load carrying system of the original building consists of concrete slabs supported by joists and RC/masonry columns. The lateral load system of the strengthened building now consists of concrete shear walls in both directions. The foundation system consists of caissons under columns with tie beams and 10 cm (4 inch) thick slab.

This building was instrumented in 1975 with 9 sensors on three levels of the building. The sensors in the building measure horizontal accelerations at the ground floor, 3rd floor, and roof level; and vertical acceleration at the ground floor (Figure 2.16). A nearby ground station (CSMIP Station No. 25091) with three sensors is the free-field site for this building.

This building yielded recorded motions during 1978 Santa Barbara earthquake. The corrected accelerations from this earthquake are presented in Figure 2.17. The peak recorded accelerations at the 3rd floor are 0.69g at channel 5 in the transverse (north-south) direction and 0.57g at channel 6 in the longitudinal (east-west) direction. The peak accelerations at the base (Ground Floor) of the building are 0.40g at channel 1 in the transverse (north-south) direction.
and 0.28g at channel 3 in the longitudinal (east-west) direction. The peak free-field accelerations in the transverse and longitudinal direction are very similar to those recorded at the base of the building: peak transverse accelerations at free-field site and base of the building are 0.35g (channel 1 in Figure 2.18) and 0.40g (channel 1 in Figure 2.17), and peak longitudinal accelerations at free-field site and base of the building are 0.29g (channel 3 in Figure 2.18) and 0.28g (channel 3 in Figure 2.17). The peak acceleration at the roof of the building in the transverse direction recorded by channel 7 is 0.99g (Figure 2.17). Note that unlike roof of other buildings considered in this investigation, roof of the Santa Barbara building is not in the horizontal plane but sloped. Therefore, accelerations recorded by channel 7 also contain contribution due to out-of-plane flexibility of the roof slab-beam system.

Figure 2.16. Sensor location in the Santa Barbara Office Building.
Figure 2.17. Accelerations recorded at the Santa Barbara Office Building during the 1978 Santa Barbara earthquake.

Figure 2.18. Accelerations recorded at the free-field site of the Santa Barbara Office Building during the 1978 Santa Barbara earthquake.
CHAPTER 3. ANALYSIS OF RECORDED MOTIONS

Recorded motions of the five selected buildings are analyzed in this chapter to extract information about the fundamental mode period and how this period may have elongated due to potential nonlinearity in the building response during strong shaking. For this purpose, transfer functions are developed in each direction of the building and analyzed for the fundamental vibration period (or frequency). The transfer function is defined as the ratio of absolute value of the Fast Fourier Transform (FFT) of the output acceleration – generally selected as that recorded at the roof level – and of the input acceleration – generally selected as that recorded at the building base in the selected direction. The fundamental vibration period is identified as the peak at the lowest frequency in the transfer function. Change in the fundamental vibration period due to possible nonlinearity in the building behavior during strong shaking is investigated by developing transfer functions for two time-segments (or time-windows) of the recorded data: an initial segment during low-level motion, and a subsequent segment with high-level motion. A significant change in the fundamental vibration period from transfer functions for these two segments of recorded data is an indicator of nonlinear behavior of the building. Note that a moving window analysis can also be used to track change in the fundamental vibration period of the building during ground shaking (Naeim, 1997). When implemented for the selected building, such moving window analysis sometimes provides trends that were not easy to interpret or explain. This more simple approach was used in this investigation.

The value of the fundamental period identified from the transfer function approach needs careful selection of the number of points in the FFT analysis, smoothing filter, and data window size, especially for buildings that may have experienced damage (or deformed beyond the linear-elastic limit) during strong ground shaking. Following is a brief discussion on selection of these parameters. Details can be found in any standard textbook on the subject (Chopra, 2007: Appendix A) or in reports specifically focused on data analysis from recorded motions (Naeim, 1997). The FFT in this investigation was implemented in MATLAB (Mathworks, 2006).

The number of points selected for the FFT analysis, $n_{pt}$, should be equal to $2^N$. If the selected data window has fewer points, the analytical procedure pads the data with zeros at the
end. Since the FFT analysis assumes that the data is periodic in nature, the zero-padding at the end may affect the results. Furthermore, the frequency resolution of the FFT results depends on the number of points selected: \( \Delta f = (1/\Delta t)/npt \) in which \( \Delta f \) is the frequency interval at which the FFT is computed, \( \Delta t \) is the time interval of the data.

The FFT results and the transfer functions computed from these results, in general, tend to be highly irregular in nature. In order to identify the real peaks, this data is often smoothed. The simple three-point, weighted-average smoothing filter of the form

\[
F_i = \frac{F_{i-1} + 2F_i + F_{i+1}}{4}
\]

is used (Naeim, 1997). This filter may be repeated as many times as necessary. However, over-smoothing may result in elimination of some important frequency identification.

Data window size selected for FFT analysis is also critical to proper system identification, especially for systems that respond in the inelastic range. For identification of the initial elastic vibration period, care must be taken to select data window as wide as possible to include several cycles of vibration of the desired mode but small enough not to extend beyond onset of yielding begins in the system.

In addition to the transfer function, 5%-damped linear-elastic response spectra in the two directions of were developed for each building site. For this purpose, accelerations recorded at the base of the building were used. Where available, the response spectra for free-field accelerations were also developed and compared with those developed for the accelerations at the building base. Differences between the spectrum for motion at the building base and free-field are indicative of the soil-structure interaction effects in the building.

**Imperial County Services Building**

The transfer functions in the longitudinal and transverse directions of the Imperial County Services building are presented in Figures 3.1 and 3.2, respectively. The identified fundamental longitudinal vibration period from an initial segment, 0 to 6 sec, of the recorded motions during the 1979 Imperial County earthquake is about 0.60 sec (Figure 3.1a). This period elongates to 1.71 sec if a later segment, 7 to 20 sec, of recorded motions is used for developing the transfer function (Figure 3.1b). The transfer function in the transverse direction (Figure 3.2) also indicates significant elongation of fundamental transverse period during the ground shaking: the
period from an initial segment is 0.41 sec (Figure 3.2a) and from later segment is 0.64 sec (Figure 3.2b). Clearly, the fundamental vibration period of this building in each direction elongated significantly during the ground shaking indicating presence of significant nonlinear action (or damage) during the ground shaking. This observation, based on transfer functions in Figure 3.1 and 3.2, is consistent with earlier damage report (ATC-9, 1984) which noted failure of columns in first story of this building during the 1979 Imperial Valley earthquake. Note that the ATC-9 report (1984) indicated that the damage in this building initiated at about 6.8 sec. Although not presented here for brevity, transfer functions for additional time windows also indicated that the vibration period remains close to 0.60 sec for first 6.5 sec of recorded motions and then suddenly increases to about 1.71 sec for all later time segments.

Figure 3.1. Transfer function for Imperial County Services Building in the longitudinal direction: (a) Time window 0 to 6 sec; and (b) Time window 7 to 20 sec.
Figure 3.2. Transfer function for Imperial County Services Building in the transverse direction: (a) Time window 0 to 6 sec; and (b) Time window 7 to 20 sec.

The 5%-damped linear-elastic spectra for the motions recorded at the base of the Imperial County Services building during the 1979 Imperial Valley earthquake are presented in Figure 3.3. Also included are the response spectra for the free-field site of this building. These results indicate that the response spectra for the building base and free-field motions are essentially the same for periods longer than 2 sec. For shorter periods, however, the free-field spectra are generally lower compared to the spectra for motions at building’s base with the difference being particularly large for periods less than about 0.5 sec. These differences are due to soil-structure interaction effects in the motions recorded at the base of the building.

Figure 3.3. 5%-damped linear elastic response spectra for motions recorded at the Imperial County Services Building: (a) Longitudinal (East-West) direction; and (b) Transverse (North-South) direction.

Sherman Oaks Commercial Building

The transfer functions in the longitudinal and transverse directions of the Sherman Oaks Commercial Building are presented in Figures 3.4 and 3.5, respectively. The identified fundamental longitudinal vibration period from an initial segment, 0 to 7 sec, of the recorded motions during the 1994 Northridge earthquake is about 2.56 sec (Figure 3.4a). This period elongates to 3.41 sec if a later segment, 9 to 20 sec, of recorded motions is used for developing the transfer function (Figure 3.4b). Clearly, the fundamental longitudinal vibration period of this building elongated significantly during the ground shaking indicating presence of nonlinear
action (or damage) during the ground shaking. The transfer functions in the transverse direction for an initial segment and a later segment (Figure 3.5) show essentially the same vibration period (2.56 sec) implying that the behavior (Figure 3.5).

The response in the longitudinal direction is dominated by the fundamental mode as apparent from a large peak at the first-mode period in the transfer function (Figure 3.4). In the transverse direction, however, several modes contribute to the response because of significant peaks in the transfer function at several modes (Figure 3.6).

Figure 3.4. Transfer function for Sherman Oaks Commercial Building in the longitudinal direction: (a) Time window 0 to 7 sec; and (b) Time window 9 to 20 sec.

Figure 3.5. Transfer function for Sherman Oaks Commercial Building in the transverse direction: (a) Time window 0 to 10 sec; and (b) Time window 11 to 20 sec.
The 5%-damped linear-elastic spectra for the motions recorded at the base of the Sherman Oaks Commercial Building during the 1994 Northridge earthquake are presented in Figure 3.6. No free-field motions are available for this building and hence corresponding response spectra are not included. These results indicate much higher spectral accelerations at short periods in the transverse direction (Figure 3.6b) compared to those in the longitudinal direction (Figure 3.6a). Such is the case because of much larger accelerations in the transverse direction: the peak acceleration in the transverse direction is 0.446g at channel 15 compared to 0.214g in the longitudinal direction at channel 13 (Figure 2.8).

![Figure 3.6. 5%-damped linear elastic response spectra for motions recorded at the Sherman Oaks Commercial Building: (a) Longitudinal (East-West) direction; and (b) Transverse (North-South) direction.](image)

**North Hollywood Hotel**

The transfer functions in the longitudinal and transverse directions of the North Hollywood Hotel are presented in Figures 3.7 and 3.8, respectively. The identified fundamental longitudinal vibration period from an initial segment, 0 to 10 sec, of the recorded motions during the 1994 Northridge earthquake is about 2.05 sec (Figure 3.7a). This period slightly elongates to 2.56 sec if a later segment, 10 to 30 sec, of recorded motions is used for developing the transfer function (Figure 3.7b). This slight elongation is not necessarily due to damage in the building but may be due to increased cracking in concrete and non-structural elements during strong shaking phase of the ground motion thus reducing the stiffness of the building. The transfer functions in the transverse direction for an initial segment and a later segment (Figure 3.8), however, led to
essentially the same vibration period (2.56 sec) implying that the behavior of this building remained within the linear elastic range in the transverse direction during the ground shaking. For reasons similar to those noted previously for the Sherman Oaks Commercial Building, higher modes contribute significantly to the response in the transverse direction (Figure 3.8) but response in the longitudinal direction occurs primarily due to the fundamental mode (Figure 3.7).

![Figure 3.7](image1.png)

*Figure 3.7. Transfer function for North Hollywood Hotel in the longitudinal direction: (a) Time window 0 to 10 sec; and (b) Time window 10 to 30 sec.*

![Figure 3.8](image2.png)

*Figure 3.8. Transfer function for North Hollywood Hotel in the transverse direction: (a) Time window 0 to 10 sec; and (b) Time window 10 to 30 sec.*

The 5%-damped linear-elastic spectra for the motions recorded at the base of the North Hollywood Hotel during the 1994 Northridge earthquake are presented in Figure 3.9. No free-field motions are available for this building and hence corresponding response spectra are not
included. These results indicate much higher spectral accelerations at short periods in the transverse direction (Figure 3.9b) compared to those in the longitudinal direction (Figure 3.9a). Such is the case because of much larger accelerations in the transverse direction: the peak acceleration in the transverse direction is 0.31g at channel 16 compared to 0.113g in the longitudinal direction at channel 14 (Figure 2.11).

Figure 3.9. 5%-damped linear elastic response spectra for motions recorded at the North Hollywood Hotel: (a) Longitudinal (East-West) direction; and (b) Transverse (North-South) direction.

**Watsonville Commercial Building**

The transfer functions in the longitudinal and transverse directions of the Watsonville Commercial Building are presented in Figures 3.10 and 3.11, respectively. The identified fundamental longitudinal vibration period from an initial segment, 0 to 4 sec, of the recorded motions during the 1989 Loma Prieta earthquake is about 0.30 sec (Figure 3.10a). This period slightly elongates to 0.38 sec if a later segment, 4 to 10 sec, of recorded motions is used for developing the transfer function (Figure 3.10b). This slight elongation is not necessarily due to damage in the building but may be due to increased cracking in concrete and increased flexibility of the soil under the building during strong shaking phase of the ground motion. The transfer functions in the transverse direction for an initial segment and a later segment (Figure 3.11) show fundamental transverse vibration period of 0.24 sec and 0.22 sec, respectively. For most practical purposed, these periods are essentially the same implying that the behavior of this building remained within the linear elastic range in the transverse direction during the ground shaking.
This slight difference may be attributed to minor errors associated with developing transfer function using the FFT approach.

Figure 3.10. Transfer function for the Watsonville Commercial Building in the longitudinal direction: (a) Time window 0 to 4 sec; and (b) Time window 4 to 10 sec.

Figure 3.11. Transfer function for the Watsonville Commercial Building in the Transverse direction: (a) Time window 0 to 4 sec; and (b) Time window 4 to 10 sec.

The 5%-damped linear-elastic spectra for the motions recorded at the base of the Watsonville Commercial Building during the 1989 Loma Prieta earthquake are presented in Figure 3.12. No free-field motions are available for this building and hence corresponding response spectra are not included. These results indicate much higher spectral accelerations at short periods in the longitudinal direction (Figure 3.12a) compared to those in the transverse direction (Figure 3.12b). Such is the case because of larger accelerations in the longitudinal
direction: the peak acceleration in the longitudinal direction is 0.36g at channel 13 compared to 0.253g in the transverse direction at channel 10 (Figure 2.14).

Figure 3.12. 5%-damped linear elastic response spectra for motions recorded at the Watsonville Commercial Building: (a) Longitudinal (East-West) direction; and (b) Transverse (North-South) direction.

**Santa Barbara Office Building**

The transfer functions in the longitudinal and transverse directions of the Santa Barbara Office Building are presented in Figures 3.13 and 3.14, respectively. The identified fundamental longitudinal vibration period from an initial segment, 0 to 3 sec, of the recorded motions during the 1978 Santa Barbara earthquake is about 0.16 sec (Figure 3.13a). This period slightly elongates to 0.22 sec for a later segment, 3 to 10 sec, of the recorded motion (Figure 3.13b). Similarly, the fundamental transverse vibration period elongates from 0.21 sec during the initial phase to about 0.27 sec during the strong-shaking phase (Figure 3.14). These slight elongations are not necessarily due to damage in the building but may be due to increased cracking in concrete and increased flexibility of the soil under the building during strong shaking phase of the ground motion.
The 5%-damped linear-elastic spectra for the motions recorded at the base of the Santa Barbara Office Building during the 1978 Santa Barbara earthquake are presented in Figure 3.15. Also included are the response spectra for the free-field site of this building. These results indicate that the response spectra for the building base and free-field motions are essentially the same for long periods. For periods shorter than about 0.5 sec, the free-field spectra are generally lower compared to the spectra for motions at building’s base. However, for periods in the range of about 0.5 sec to 2 sec, the free-field spectra are generally much higher compared to the spectra for motions at building’s base. These differences are again due to soil-structure interaction effects in the motions recorded at the base of the building.
Figure 3.15. 5%-damped linear elastic response spectra for motions recorded at the Santa Barbara Office Building: (a) Longitudinal (East-West) direction; and (b) Transverse (North-South) direction.
CHAPTER 4. INTERPOLATION OF RECORDED MOTIONS

Buildings are typically instrumented at limited number of floors, and motions of the remaining (or non-instrumented) floors are estimated by interpolation procedure. Typically, a piece-wise cubic polynomial interpolation (PWCP) procedure is used for conventional buildings (Naeim, 1997; De la Llera and Chopra, 1998; Goel, 2005, 2007; Limongelli, 2003) and a combination of cubic-linear interpolation is recommended for base-isolated buildings (Naeim, et al., 2004). It is generally believed that the PWCP procedure provides reasonable estimates of motions at non-instrumented floors (Naeim, 1997; Naeim et al., 2004; De la Llera and Chopra, 1998). Presented in this chapter is the background for the PWCP procedure followed by the interpolated motions for each of the five buildings selected in this investigation.

Piece-Wise Cubic Polynomial Interpolation

Let an $N$-Story building be instrumented at $J$ locations. Also let $r_j$ be the response – displacement or acceleration – recorded by the $j$th sensor located at height $h_j$ from the building base, and $r$ be the desired response at height $h$ (or non-instrumented location) of the building. The response $r$ is to be computed by interpolation of recorded responses $r_j$. Following is the theoretical background of the commonly used PWCP procedure.

The response over height of the building be approximated by a polynomial of order $k$

$$r(h) = \sum_{i=0}^{k} c_i h^i \quad (4.1)$$

in which $c_i$ is the $i$th coefficient of the polynomial. This polynomial equation involves $k+1$ unknown coefficients, which can be computed uniquely if motions are available at $k+1$ locations over the building height. Once the unknown coefficients have been computed, motions at any intermediate (or non-instrumented) floor can be computed by Equation 4.1. However, accurate estimate of responses over the building height using Equation 4.1 requires polynomial of very high order.
An alternative approach is to sub-divide the interpolation interval to several smaller sub-intervals and fit a piece-wise cubic polynomial to each sub-interval. Let us consider a building with sensors at $J$ locations and sub-divided into $J - 1$ sub-intervals for buildings with sensors at the base and the roof. The response, $r$, at height, $h$, located in the $j$th sub-interval is given by

$$r(h) = a_j(h - h_j)^3 + b_j(h - h_j)^2 + c_j(h - h_j) + d_j$$

(4.2)

in which $a_j$, $b_j$, $c_j$, and $d_j$ are the constants for the cubic-polynomial to be fitted in the $j$th sub-interval. Since Equation 4.2 for each sub-interval involves four constants, $4(J - 1)$ constants are needed to completely define the response of a building with $J - 1$ sub-intervals, which in turn implies that $4(J - 1)$ equations are required to uniquely solve for these constants.

Since recorded response is available at both ends of each sub-interval, forcing Equation 4.2 to match the recorded response at these locations provides $2(J - 1)$ equations. The remaining $2(J - 1)$ equations may be obtained by forcing continuity conditions at junctions of two adjacent sub-intervals and utilizing the boundary conditions (or known values of derivatives of the response) at the base and top of the building. Since the derivatives with respect to space of the response at the bottom and top of the building are usually not available, one of the most commonly used boundary condition is the “not-a-knot” condition. With this boundary condition, the remaining $2(J - 1)$ equations are obtained as: (1) $2(J - 2)$ equations by forcing the first and second derivatives of the response to be equal at $(J - 2)$ junctions of the $(J - 1)$ sub-intervals; (2) one equation by forcing the third derivative of the response to be equal at top of the first sub-interval and bottom of the second-sub-interval; and (3) one equation by forcing the third derivative of the response to be equal at top of the last-but-one sub-interval and bottom of the last sub-interval.

The computation of the constants using the aforementioned “not-a-knot” end condition involves solution of a tri-diagonal system of linear equations. For this purpose, let us re-cast Equation 4.2 as
\[ r(h) = a_j \bar{w}_j + b_j w_j + c_j \frac{(\Delta h_j)^2}{6} \left( \bar{w}_j^3 - \bar{w}_j \right) + d_j \frac{(\Delta h_j)^2}{6} \left( w_j^3 - w_j \right) \]  

(4.3)

in which \( w_j = (h - h_j) / (h_{j+1} - h_j) \); \( \bar{w}_j = (h_{j+1} - h) / (h_{j+1} - h_j) \); \( \Delta h_j = h_{j+1} - h_j \); and \( a_j \), \( b_j \), \( c_j \), and \( d_j \) are the cubic-polynomial constants for the \( j \)th sub-interval.

The polynomial constants for each of the \( J - 1 \) sub-intervals are \( a_j = r_j \), \( b_j = r_{j+1} \), \( c_j = \sigma_j \), and \( d_j = \sigma_{j+1} \). The \( J \) values of \( \sigma_j \) for piece-wise cubic polynomial interpolation with “not-a-knot” end conditions are computed from the following set of linear equations (Beatson, 1986):

\[-\overline{\theta}_j \sigma_1 + \sigma_2 - \theta_1 \sigma_3 = 0
\]

\[ \theta_{j-1} \sigma_{j-1} + 2 \sigma_j + \overline{\theta}_{j-1} \sigma_{j+1} = \frac{6}{h_{j+1} - h_j - 1} \left[ \left( \frac{r_{j+1} - r_j}{h_{j+1} - h_j} \right) - \left( \frac{r_j - r_{j-1}}{h_j - h_{j-1}} \right) \right] \quad j = 2,3,...,J - 1 \]  

(4.4)

\[-\overline{\theta}_{j-2} \sigma_{j-2} + \sigma_{j-1} - \theta_{j-2} \sigma_j = 0 \]

in which

\[ \theta_j = \frac{h_{j+1} - h_j}{h_{j+2} - h_j} \quad j = 1,2,...,J - 2; \quad \overline{\theta}_j = \frac{h_{j+2} - h_{j+1}}{h_{j+2} - h_j} \quad j = 1,2,...,J - 2 \]  

(4.5)

In the matrix form, solution for \( J \) values of \( \sigma_j \) involves solving the following system:

\[ A \sigma = b \]  

(4.6)

in which \( \sigma = [\sigma_1 \sigma_2 \sigma_3 \cdots \sigma_{J-2} \sigma_{J-1} \sigma_J]^T \),

\[ A = \begin{bmatrix}
-\overline{\theta}_1 & 1 & -\theta_1 & 0 \\
\theta_1 & 2 & \overline{\theta}_1 & 0 \\
0 & \theta_2 & 2 & \overline{\theta}_2 \\
& & \ddots & \ddots \\
\theta_{J-3} & 2 & \overline{\theta}_{J-3} & 0 \\
0 & \theta_{J-2} & 2 & \overline{\theta}_{J-2} \\
0 & -\overline{\theta}_{J-2} & 1 & -\theta_{J-2}
\end{bmatrix} \]  

(4.7)
\[ b = \begin{pmatrix}
0 \\
\frac{6}{h_3 - h_1} \left( \frac{r_3 - r_2}{h_3 - h_2} - \frac{r_2 - r_1}{h_2 - h_1} \right) \\
\frac{6}{h_4 - h_2} \left( \frac{r_4 - r_3}{h_4 - h_3} - \frac{r_3 - r_2}{h_3 - h_2} \right) \\
\vdots \\
\frac{6}{h_{J-1} - h_{J-3}} \left( \frac{r_{J-1} - r_{J-2}}{h_{J-1} - h_{J-2}} - \frac{r_{J-2} - r_{J-3}}{h_{J-2} - h_{J-3}} \right) \\
\frac{6}{h_J - h_{J-2}} \left( \frac{r_J - r_{J-1}}{h_J - h_{J-1}} - \frac{r_{J-1} - r_{J-2}}{h_{J-1} - h_{J-2}} \right) \\
0
\end{pmatrix} \] (4.8)

Once \( \sigma_j \) have been computed from Equation 4.6, constants \( c_j \) and \( d_j \) can be determined \((c_j = \sigma_j \) and \( d_j = \sigma_{j+1} \)) and motions at any non-instrumented floors can be estimated from Equation 4.3. The formulation presented here has been verified against the spline function in MATLAB (MathWorks, 2006) with “not-a-knot” end conditions.

**Interpolated Motions**

Interpolated motions – total floor displacements and total floor accelerations – for each of the five selected building in the longitudinal (east-west) and transverse direction (north-south) are presented in Figure 4.1 to 4.10. Also included in these figures are the story drifts computed from interpolated floor displacements. The interpolation procedure is applied to motions at the center of the building floor plan. For each of the selected buildings, sensors measuring floor motions in the longitudinal direction already measure motions at the center of the building. In the transverse direction, however, there may not always be a sensor at the center of the floor plan. Therefore, sensors located on two sides of the floor and measuring transverse motions are used to compute the motion at the center of the floor plan assuming in-plane rigidity of the floor slab. The interpolated motions are used in later chapters of this report to compute the peak relative floor displacements and story drifts which are then compared with the values estimated from various nonlinear static procedures.
Figure 4.1. Motions at center of the Imperial County Services Building in the longitudinal (east-west) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.2. Motions at center of the Imperial County Services Building in the transverse (north-south) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.3. Motions at center of the Sherman Oaks Commercial Building in the longitudinal (east-west) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.4. Motions at center of the Sherman Oaks Commercial Building in the transverse (north-south) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.5. Motions at center of the North Hollywood Hotel in the longitudinal (east-west) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.6. Motions at center of the North Hollywood Hotel in the transverse (north-south) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.7. Motions at center of the Watsonville Commercial Building in the longitudinal (east-west) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.

Figure 4.8. Motions at center of the Watsonville Commercial Building in the transverse (north-south) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
Figure 4.9. Motions at center of the Santa Barbara Office Building in the longitudinal (east-west) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.

Figure 4.10. Motions at center of the Santa Barbara Office Building in the transverse (north-south) direction: (a) Floor displacements, (b) Story drifts, and (c) Floor accelerations.
CHAPTER 5. ANALYTICAL MODELS

Three-dimensional analytical models of the selected buildings were developed using the structural analysis software Open System for Earthquakes Engineering Simulation (OpenSees) (McKenna and Fenves, 2001). Two models were developed for each building: linearly-elastic model for computing the mode shapes and frequencies (or vibration periods), and a nonlinear model for pushover analysis. The beams, columns, and shear walls in the linear elastic model were based on effective section properties recommended in the FEMA-356 document (ASCE-2000). The rigid-end offset at connection between beam and columns were included in the model where appropriate. The beams, columns, and shear walls were modeled using elasticBeamColumn element in OpenSees.

The beams, columns, and shear walls in the nonlinear model were modeled either with beamWithHinges or nonlinearBeamColumn element in OpenSees. Both elements used fiber sections containing confined concrete, unconfined concrete, and steel reinforcing bars to model the axial-flexural behavior, whereas linear-elastic force-deformation was assumed for the shear and torsional behavior. The stress-strain behavior of concrete, both confined and confined, was modeled with Concrete04 material in OpenSees (Figure 5.1a). The Concrete04 material model differs from the traditionally used Concrete01 material model in OpenSees. The Concrete04 material model completely losses strength immediately after the crushing strain whereas Concrete01 material model exhibits residual strength equal to the concrete strength at the crushing strain for all values of strains larger than the crushing strain. The crushing strain of the unconfined concrete was selected to be equal to 0.004 and that for confined concrete was selected to be that corresponding to the rupture of confining steel using the well established Mander model (Mander et al., 1988; Priestly et al., 1998). The stress-strain behavior of steel was modeled with ReinforcingSteel material in OpenSees (Figure 5.1b). Further details of the
material models are available in McKenna and Fenves (2001). The strength of concrete and steel was selected based on the values specified in the structural drawings.

![Figure 5.1. Material models used for nonlinear analysis.](image)

For two of the five selected buildings – Watsonville Commercial Building and Santa Barbara Office Building – the foundation flexibility was expected to significantly influence the response during strong ground shaking because both of these low-rise buildings contained longitudinal and transverse shear walls. The foundation flexibility was included in analytical models of these buildings by attaching six linear springs – three along the x-, y-, and z-translation, two about the x- and y- rocking, and one about the z-torsion – at the base as per the FEMA-356 recommendations for foundation flexibility modeling (ASCE, 2000).

It is of interest to note that the analytical models developed and used in this investigation are based on generally acceptable engineering practice. The analytical models were not specifically calibrated to provide targeted values of either the vibration periods or the response history.

The fundamental mode shapes in the longitudinal and transverse direction computed from eigen analysis of the linear-elastic model are presented in Figures 5.2 to 5.7 for each of the five selected buildings. Also included in these figures are the fundamental vibration periods in the two directions. Following is a brief discussion of how the fundamental vibration periods from eigen analysis of the analytical models compare with the previously identified periods from recorded motions.
The fundamental vibration period of the Imperial County Services building in the longitudinal direction is about 1.2 sec (Figure 5.2a). This period correlates well with the longitudinal period of 1.0 sec computed in an earlier report (ATC-9, 1984). The longitudinal period identified from motions recorded during the 1979 Imperial Valley earthquake was 0.602 sec from an early segment – 0 to 6 sec – and 1.71 sec from a segment during strong ground shaking – 7 to 20 sec – of the recorded motion (Figure 3.1). The computed longitudinal period of about 1.2 sec (Figure 5.2a) is between the two identified values (Figure 3.1). Recall that the Imperial County Services Building suffered significant damage during the 1979 Imperial Valley earthquake and the damage was believed to be initiated in the longitudinal direction. Therefore, it may not be possible to obtain a good match between computed longitudinal period and the period identified from recorded motion.

The fundamental vibration period of the Imperial County Services Building in the transverse direction is about 0.44 sec (Figure 5.2b). This period correlated well with the period of 0.41 sec identified from an early segment of the motion of this building recorded during the 1979 Imperial Valley earthquake (Figure 3.2) and a period of 0.40 sec computed in the ATC-9 report (ATC-9, 1984). The transverse mode shape also exhibits coupling between transverse and torsional motions. This is the case due to unsymmetrical placement of shear walls in the first story (see Figure 2.3). Additional asymmetry also occurs because the shear-wall on the west side of the building has openings whereas the shear wall on the east side has no openings from 2nd floor to roof. The recorded motions presented in Figure 2.4 also indicate presence of torsional motions in this building. This becomes apparent from different motions on two sides of a floor, e.g., motions recorded by sensor 1 on the west-side and sensor 3 on the east-side of the roof (Figure 2.4).
The fundamental vibration period of the Sherman Oaks Commercial Building is 2.67 sec in the longitudinal direction (Figure 5.3a) and 2.94 sec in the transverse direction (Figure 5.3b). The computed longitudinal period of 2.67 sec is very close to the period of 2.56 sec identified from an early segment of the recorded motions (Figure 3.4a). The computed transverse period of 2.94 sec is slightly longer than the period of 2.56 identified from recorded motions in this direction (Figure 3.5).

The fundamental transverse mode shape of the Sherman Oaks Commercial Building also exhibits coupling between transverse and torsional motions (Figure 5.3b). This coupling was also found based on recorded motions presented previously in Figure 2.8. In particular, recorded motions on two sides of the building in the transverse direction differed significantly indicating the presence of torsional motions.
Figure 5.3. Fundamental mode shape and period of the Sherman Oaks Commercial Building: (a) Longitudinal direction, and (b) Transverse direction.

The fundamental vibration period of the North Hollywood Hotel is 2.4 sec in the longitudinal direction (Figure 5.4a) which is in the range of 2.05 sec and 2.56 sec identified previously from recorded motions (Figure 3.7). The fundamental period computed in the transverse direction is 2.8 sec (Figure 5.4b) which is slightly longer than the value of 2.56 sec identified from recorded motions (Figure 3.8).

The recorded motion presented earlier in Figure 2.11 indicated the presence of coupling between transverse and torsional motions. While the fundamental transverse mode (Figure 5.4b) does not exhibit torsional coupling, the second transverse mode with a vibration period of about 0.8 sec does show coupling between transverse and torsional modes (Figure 5.5b). Therefore, the coupling between transverse and torsional motions in Figure 2.11 appears to be due to significant contribution of the 2nd transverse mode. Recall that higher mode contribution was found to be significant in the transverse direction of this building based on the transfer function analysis (Figure 3.8) As was the case for the fundamental mode in the longitudinal direction (Figure 5.4a), the 2nd longitudinal mode also exhibits no torsional coupling (Figure 5.5a).
The fundamental period of the Watsonville Commercial Building in the longitudinal (east-west) direction is about 0.31 sec (Figure 5.6a) which is between the periods of 0.30 sec and 0.38 sec identified from two segments of longitudinal motions recorded for this building during the 1989 Loma Prieta earthquake (Figure 3.10). The fundamental mode shape in the longitudinal
direction also exhibits significant torsional coupling (Figure 5.6a). Such is the case due to open first story on south side of the building (see figure 2.13). The torsional coupling was also apparent in recorded motions presented previously in Figure 2.14. The fundamental vibration period of the Watsonville Commercial Building in the transverse direction is about 0.28 sec (Figure 5.6b) which is close to the values of 0.24 sec and 0.22 sec identified in Figures 3.11 from recorded motions.

![Figure 5.6](image)

Figure 5.6. Fundamental mode shape and period of the Watsonville Commercial Building: (a) Longitudinal (east-west) direction, and (b) Transverse (north-south) direction.

The mode shapes of the Watsonville Commercial Building presented in Figure 5.6 also indicate motion at the building’s base; the motion at the base is visible to a larger extent in the transverse mode (Figure 5.6b) compared to the longitudinal mode (Figure 5.6a). The motion at the base is due to the presence of flexible soil springs that were included to model the soil flexibility for this building.

The fundamental vibration period of the Santa Barbara Office Building in the longitudinal direction is 0.15 sec (Figure 5.7a) which matches quite well with the period identified from an early segment of the motions recorded during the 1978 Santa Barbara earthquake (Figure 3.13a). Similarly the fundamental vibration period in the transverse direction is 0.18 sec (Figure 5.7b) which is also close to the period of 0.2 sec identified from recorded motions (Figure 3.14a). Note that the vibration periods of 0.22 sec and 0.27 sec in the
longitudinal and transverse directions identified from strong shaking potion of recorded motions (Figures 3.13b and 3.14b) are longer than the periods computed in Figures 5.7a and 5.7b. The longer periods identified from recorded motions are due to softening of the soil during strong shaking phase of ground motion.

Figure 5.7. Fundamental mode shape and period of the Santa Barbara Office Building: (a) Longitudinal (east-west) direction, and (b) Transverse (north-south) direction.
CHAPTER 6. CURRENT NONLINEAR STATIC PROCEDURES

Nonlinear static procedure (NSP) in the FEMA-356, ATC-40, FEMA-440, and ASCE-41 documents require development of a pushover curve which is defined as the relationship between the base shear and lateral displacement of a control node. The height-wise distributions of lateral loads for pushover analysis is typically selected from: (1) Equivalent lateral force (ELF) distribution: \( s_j^* = m_j h_j^k \) (the floor number \( j = 1, 2 \ldots N \)) where \( s_j^* \) is the lateral force and \( m_j \) the mass at \( j \)th floor, \( h_j \) is the height of the \( j \)th floor above the base, and the exponent \( k = 1 \) for fundament period \( T_1 \leq 0.5 \text{ sec} \), \( k = 2 \) for \( T_1 \geq 2.5 \text{ sec} \); and varies linearly in between; (2) Fundamental mode distribution: \( s_j^* = m_j \phi_{j1} \) where \( \phi_{j1} \) is the fundamental mode shape component at the \( j \)th floor; and (3) Response Spectrum Analysis (RSA) distribution: the vector of lateral forces \( s^* \) is defined by the lateral forces back-calculated from the story shears determined by linear response spectrum analysis of the structure including sufficient number of modes to capture 90% of the total mass; and (4) “Uniform” distribution: \( s_j^* = m_j \) in which \( m_j \) is the mass and \( s_j^* \) is the lateral force at \( j \)th floor. The FEMA-356 NSP requires development of the pushover curve for two height-wise distributions of lateral forces: one selected from the first three of the aforementioned distributions and the second selected as the “Uniform” distribution. The ATC-40, FEMA-440, and ASCE-41 NSPs require development of the pushover curve only for the fundamental mode distribution.

The structure is pushed statically to a target displacement at the control node to check for the acceptable structural performance. The NSP in the FEMA-356, FEMA-440, ATC-40, and ASCE-41 documents differ primarily in computation of this target displacement. These methods are summarized in this chapter.
FEMA-356 Coefficient Method

The target displacement in the Coefficient Method (CM), specified in the FEMA-356 document (ASCE, 2000) is computed from:

\[ \delta_i = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \]  

(6.1)

where \( S_a \) = Response spectrum acceleration at the effective fundamental vibration period and damping ratio of the building under consideration; \( g \) = Acceleration due to gravity; \( T_e \) = Effective fundamental period of the building in the direction under consideration computed by modifying the fundamental vibration period from elastic dynamic analysis, e.g., eigen-value analysis, \( T_i \), by:

\[ T_e = T_i \sqrt{\frac{K_i}{K_e}} \]  

(6.2)

in which \( K_i \) is the elastic stiffness of the building and \( K_e \) is the effective stiffness of the building obtained by idealizing the pushover curve as a bilinear relationship; \( C_0 \) = Modification factor that relates the elastic response of a Single-Degree-of-Freedom (SDF) system to the elastic displacement of the Multi-Degree-of-Freedom (MDF) building at the control node taken as the first mode participation factor or selected from tabulated values in FEMA-356; \( C_1 \) = Modification factor that relates the maximum inelastic and elastic displacement of the SDF system computed from

\[ C_1 = \begin{cases} 
1.0; & T_e \geq T_s \\
1.0 + \frac{(R-1) T_s / T_e}{R}; & T_e < T_s \\
1.5; & T_e < 0.1s 
\end{cases} \]  

(6.3)

in which \( R \) is the ratio of elastic and yield strengths and \( T_s \) is the corner period where the response spectrum transitions from constant pseudo-acceleration to constant pseudo-velocity; \( C_2 \) = Modification factor to represent the effects of pinched hysteretic shape, stiffness
degradation, and strength deterioration selected either from tabulated values depending on the framing system (see FEMA-356 for details of various framing systems) and the performance level or taken as one for nonlinear analysis; and \( C_3 \) = Modification factor to represent increased displacement due to P-delta effects computed from

\[
C_3 = \begin{cases} 
1.0; & \alpha \geq 0 \\
1.0 + \frac{\vert \alpha \vert (R-1)^{3/2}}{T_e}; & \alpha < 0 
\end{cases} 
\]  
(6.4)

in which \( \alpha \) is the ratio of the post-yield stiffness to effective elastic stiffness. In Equations 6.3 and 6.4, \( R \) is defined as

\[
R = \frac{S_a}{V_y/W} C_m 
\]  
(6.5)

where \( V_y \) is the yield strength of the building estimated from the idealized nonlinear force-displacement (or pushover) curve of the building, \( W \) is the effective seismic weight, and \( C_m \) is the effective modal mass factor for the fundamental mode of the building.

**ATC-40 Capacity Spectrum Method**

The target displacement in Capacity Spectrum Method (CSM) specified in the ATC-40 document (ATC-40, 1997) is computed from:

\[
\delta_i = C_0 S_d(T_{eq}, \zeta_{eq}) 
\]  
(6.6)

where \( S_d(T_{eq}, \zeta_{eq}) \) is the maximum displacement of a linearly-elastic SDF system with equivalent period, \( T_{eq} \), and equivalent damping ratio, \( \zeta_{eq} \) given by:

\[
T_{eq} = T_o \sqrt{\frac{\mu}{1+\alpha \mu - \alpha}}; \quad \zeta_{eq} = \zeta_o + \kappa - \frac{1}{\pi \mu (1+\alpha \mu - \alpha)} 
\]  
(6.7)

in which \( T_o \) is the initial period of vibration of the system, \( \alpha \) is the post-yield stiffness ratio, \( \mu \) is the maximum displacement ductility ratio, and \( \kappa \) is the adjustment factor to approximately
account for changes in hysteretic behavior of reinforced concrete structure. The ATC-40 document defines three types of hysteretic behaviors – Type A with stable, reasonably full hysteretic loops; Type C with severely pinched and/or degraded loops; and Type B between Types A and C – and provides equations for computing $\kappa$ for each of the three types of hysteretic behavior.

Since the equivalent linearization procedure requires prior knowledge of the displacement ductility ratio (Equation 6.7), ATC-40 document describes three iterative procedures: Procedures A, B, and C. Procedures A and B are the most transparent and convenient for programming, whereas Procedure C is purely a graphical method. Details of these procedures are available in ATC-40 document and are not presented here for brevity.

**ASCE-41 Coefficient Method**

The ASCE-41 CM is based on the improvements to the FEMA-356 CM proposed in the FEMA-440 document (ATC, 2005). The target displacement in the ASCE-41 CM (ASCE, 2007) is computed from:

$$\delta_t = C_0 C_1 C_2 S a \frac{\tau_e^2}{4\pi^2} g$$  \hspace{1cm} (6.8)

The coefficient $C_1$ is given by

$$C_1 = \begin{cases} 
1.0; & T_e > 1.0 \text{s} \\
1.0 + \frac{R - 1}{a T_e^2}; & 0.2 \text{s} < T_e \leq 1.0 \text{s} \\
1.0 + \frac{R - 1}{0.04a}; & T_e \leq 0.2 \text{s}
\end{cases}$$  \hspace{1cm} (6.9)

in which $a$ is equal to 130 for site class A and B, 90 for site class C, and 60 for site classes D, E, and F (see ASCE-41 for details of various site classes), respectively. The coefficient $C_2$ is given by
Finally, ASCE-41 CM has dropped the coefficient $C_3$ but imposed a limitation on strength to avoid dynamic instability. This limitation on strength is specified by imposing a maximum limit on $R$ given by

$$R_{\text{max}} = \frac{\Delta_d}{\Delta_y} + \frac{\left| \alpha_e \right|^h}{4}; \quad h = 1.0 + 0.15 \ln \left( T_e \right)$$  \hspace{1cm} (6.11)

in which $\Delta_d$ is the deformation corresponding to peak strength, $\Delta_y$ is the yield deformation, and $\alpha_e$ is the effective negative post-yield slope given by

$$\alpha_e = \alpha_{p-\Delta} + \lambda \left( \alpha_2 - \alpha_{p-\Delta} \right)$$  \hspace{1cm} (6.12)

where $\alpha_2$ is the negative post-yield slope ratio defined in Figure 6.1, $\alpha_{p-\Delta}$ is the negative slope ratio caused by $P-\Delta$ effects, and $\lambda$ is the near-field effect factor given as 0.8 for $S_1 \geq 0.6$ and 0.2 for $S_1 < 0.6$ ($S_1$ is defined as the 1-second spectral acceleration for the Maximum Considered Earthquake). The $\alpha_2$ slope includes $P-\Delta$ effects, in-cycle degradation, and cyclic degradation.

![Figure 6.1. Idealized force-deformation curve in ASCE-41.](image-url)
FEMA-440 Capacity Spectrum Method

The target displacement in FEMA-440 CSM (ATC, 2005) is computed from:

\[ \delta_t = C_0 S_d(T_{\text{eff}}, \zeta_{\text{eff}}) \]  \hspace{1cm} (6.13)

where \( S_d(T_{\text{eff}}, \zeta_{\text{eff}}) \) is the maximum displacement of a linearly-elastic SDF system with effective period, \( T_{\text{eff}} \), and effective damping ratio, \( \zeta_{\text{eff}} \). The improved FEMA-440 CSM includes improved expressions, compared to the ATC-40 CSM, to determine the effective period and effective damping developed by Guyader and Iwan (2006). Consistent with the original ATC-40 procedure, three iterative procedures for estimating the target displacement are also outlined. Finally, a limitation on the strength is imposed to avoid dynamic instability (Equation 6.8).

The improved formulas for effective period and damping ratio in the FEMA-440 document are:

\[
T_{\text{eff}} = \begin{cases} 
0.2(\mu - 1)^2 - 0.038(\mu - 1)^3 + 1 & \mu < 4.0 \\
0.28 + 0.13(\mu - 1) + 1 & 4.0 \leq \mu \leq 6.5 \\
0.89 \left( \frac{\mu-1}{1+0.05(\mu-2)} \right)^{-1} + 1 & \mu > 6.5 
\end{cases}
\]  \hspace{1cm} (6.14a)

\[
\zeta_{\text{eff}} = \begin{cases} 
4.9(\mu - 1)^2 - 1.1(\mu - 1)^3 + \zeta_o & \mu < 4.0 \\
14.0 + 0.32(\mu - 1) + \zeta_o & 4.0 \leq \mu \leq 6.5 \\
19 \left[ \frac{0.64(\mu - 1) - 1}{0.64(\mu - 1)^2} \right] \left( \frac{T_{\text{eq}}}{T_o} \right)^2 + \zeta_o & \mu > 6.5 
\end{cases}
\]  \hspace{1cm} (6.14b)

These formulas apply for periods in the range of 0.2 and 2.0s. The FEMA-440 document also provides formulas with constants \( A \) to \( L \) that are specified depending on the force-
deformation relationships (bilinear, stiffness-degrading, strength-degrading) and the post-yield stiffness ratio, $\alpha$; these formulas are not included here brevity.
CHAPTER 7. EFFECTS OF MODELING ASSUMPTIONS ON PUSHOVER CURVE

It is useful to investigate the effects of various assumptions in analytical model of a building on its nonlinear behavior or pushover curve. For this purpose, investigated first is how two different approaches to modeling nonlinear beam-column elements influence the pushover curve of the building. Subsequently, effect of material strength – nominal strengths specified on structural drawings versus expected strength – on the pushover curve is examined. Finally, the differences in pushover curves from different computer programs are investigated. The investigation in this chapter is limited to nonlinear behavior of only one building – the Imperial County Services Building.

Effects of Modeling Assumptions

In order to examine the effects of different approaches to modeling nonlinear beam-column elements, the pushover curves are developed by two nonlinear analyses. The first analysis, implemented in the computer program Capacity Analysis Pushover Program (CAPP) (Chadwell, 2007), utilized section moment-curvature relationship translated into a moment-rotation relationship using an assumed plastic hinge length for modeling nonlinear beams, columns, and shear-walls. The plastic length was found assuming double-curvature bending in proportion to the member’s overstrength. The overstrength was defined as the ultimate moment divided by the yield moment. In the second analysis, implemented in OpenSees (McKenna and Fenves, 2001), a fiber section was utilized for modeling nonlinear beams, columns, and shear-walls. The moment-curvature relationship utilized in \textit{CAPP} was terminated when the first crushing in concrete or rupture in longitudinal steel occurred and was calculated with axial forces equal to that due to gravity loads.

The pushover curves in the longitudinal direction of the Imperial County Services Building from the two approaches – fiber-section in \textit{OpenSees} and moment-curvature in \textit{CAPP} – are compared in Figure 7.1. This comparison shows that the two approaches provide similar behavior of the building during initial phase of the pushover analysis as apparent from the two curves being essentially identical for roof displacement up to 5 cm. The moment-curvature approach, however, leads to significant loss of strength at much lower roof displacement compared to the fiber-section approach; significant loss of strength in the models with moment-
curvature and fiber-section approaches occurred at roof displacement of about 5 cm and 14 cm, respectively. The early loss of strength in the model with moment-curvature approach occurs due to assumption that the section completely loses strength at the first crushing of concrete or fracture in longitudinal steel. The fiber-section in the OpenSees model continues to provide strength (or resistance) even after the first crushing of concrete or fracture of longitudinal steel due to continued resistance provided by the remaining fibers (both concrete and steel) in the section.

![Figure 7.1. Comparison of pushover curves in longitudinal direction of the Imperial County Services Building from analytical models using fiber-section and section moment-curvature approaches.](image)

The results presented in Figure 7.1 indicate that analytical models that are based on concentrated plastic hinges at the two ends of the beam-column element, with properties of the plastic hinges defined from moment-curvature relationship, may lead to prediction of building failure at much lower roof displacement compared to analytical models that utilize fiber-sections to model the nonlinear behavior in beam-column elements. As noted previously, this early loss in strength is due to the assumption in the model with concentrated plastic hinges that the section loses strength at the first crushing of concrete or fracture of longitudinal steel. In reality, the section should be able to provide additional resistance, and hence strength, even after first crushing of concrete or fracture of longitudinal steel this, in turn, suggests that analytical models based on fiber-section model may provide a more realistic estimate of failure deformation.
Effects of Material Strength

In order to investigate the effects of material strength, pushover curves are developed for two sets of material strengths: the nominal strengths equal to those specified on structural drawings and expected material strengths. The nominal compressive strength for concrete specified on structural drawings of the Imperial County Services Building was 35 MPa (5 ksi). The nominal values of yield strength and ultimate failure strength of reinforcing steel was 275 MPa (40 ksi) and 412.5 MPa (60 ksi) respectively. The expected strengths of concrete and reinforcing steel were assumed to be 1.5 times and 1.25 times, respectively, the nominal strengths. The factors to compute the expected material strengths of concrete and reinforcing steel were selected based on the FEMA-356 recommendations (ASCE, 2000).

The pushover curves for the two sets of material strength are presented in Figure 7.2. These results indicate higher building strength of pushover curves based on the expected strength. This is to be expected because of higher values of expected material strengths compared to the nominal values. Furthermore, the building strength increases by a factor of about 1.2 in the pushover curve based on expected material strengths compared to that based on nominal material strengths. This increase in the building strength appears to be correlated to the increased value of the steel expected strength relative to its nominal strength.

![Figure 7.2. Comparison of curves in longitudinal direction of the Imperial County Services Building for nominal and expected material strengths.](image)

In order to understand the reason for higher strength in Figures 7.2a and 7.2b, pushover curve for another set of material properties is developed: the compressive strength of concrete is
selected to be equal to the nominal strength, and yield and ultimate strengths selected to be equal to the expected values of these strengths. The comparison presented in Figure 7.3 indicates that the increase in strength is primarily due to increased expected strength of steel. This result is not unexpected. It is of interest to note that the higher expected strength of steel does not affect the initial stiffness of the pushover curve as apparent from almost identical initial stiffness of the two pushover curves. The increased stiffness in the pushover curve for expected strengths of concrete and steel, therefore, is almost entirely due to increased strength of concrete.

Figure 7.3. Comparison of curves in longitudinal direction of the Imperial County Services Building for nominal strength of both concrete and steel, expected strength of both steel and concrete, and expected strength of steel alone.

**Pushover Curves from Different Computer Programs**

In order to understand if different computer programs would provide similar pushover curves, pushover analysis of the Imperial County Services Building in the longitudinal direction was implemented in three different computer programs: *OpenSees* (McKenna and Fenves, 2001), *CAPP* (Chadwell, 2007), and *CANNY* (Li, 2004). These analyses were implemented independently by three different investigators based on the same set of structural drawings and material properties. The three computer programs, however, may have utilized different approaches to modeling nonlinear behavior of beam-column element or material stress-strain relationships. No attempt was made in this phase of the investigation to synchronize the modeling approaches or material stress-strain in the three computer programs.
The results presented in Figure 7.4 indicate that the three computer programs provide pushover curves with similar initial stiffness for roof displacement of about 2 cm. For larger roof displacements, however, the CANNY provides the pushover curve that has much lower stiffness compared to the pushover curves from OpenSees and CAPP models. Furthermore, the pushover curve from CANNY indicates lower strength of the building compared to the pushover curves from OpenSees and CAPP models. While pushover curves from OpenSees and CAPP models exhibit slightly negative post-yield stiffness, as apparent from gradual loss of strength after yield strength, pushover curve from CAPP does not indicate negative-post yield stiffness. Finally, the yield strength of the pushover curve from CANNY is less than that of pushover curves from OpenSees and CAPP.

The differences in the stiffness of pushover curves from the three computer programs appear to be due to differences in the moment-curvature relationships. Therefore, summarized next is how moment-curvature relationships were used in the three programs. The computer program OpenSees did not explicitly use the moment-curvature relationship of the nonlinear beam-column element developed prior to the pushover analysis but rather the moment-curvature relationship of various sections along the member length (at integration points) was developed during the pushover analysis. The computer program CAPP utilized the moment-curvature relationship developed prior to the pushover analysis then translated to a moment-rotation relationship. This relationship was developed for section axial force equal to that due to gravity loads alone. The moment-curvature relationship was idealized as a multi-linear curve with initial linear elastic portion from zero moment to the effective yield moment with termination of the moment-curvature relationship occurring at first crushing of concrete or rupture in reinforcing steel. The CANNY computer program also utilized the moment-curvature relationship. However, the moment-curvature relationship was idealized as tri-linear curve with first linear curve between zero moment and cracking moment, second linear curve between cracking moment and yield moment, and a final linear curve from yield moment onwards. Typically, the stiffness of the second curve is about 20 to 30% of the first curve and that of the final curve is about 5% of the first curve.

The above description of moment curvature relationship indicates that the lower stiffness of the pushover curve from the CANNY model is due to much softer second portion of the tri-
linear idealization used in this program. The lack of negative-post yield stiffness in the pushover curve from the CANNY model is due to the positive slope, albeit small, of the last segment of the idealized moment-curvature relationship and because P-Delta effects due to gravity loads were not considered. Although reason for lower yield strength of the pushover curve from the CANNY model is not entirely clear, it appears to be due differences in stress-strain relationships of concrete and steel compared to those used on the OpenSees and CAPP models. As mentioned previously, differences between pushover curves from OpenSees and CAPP models are due to different approaches to modeling nonlinear behavior of beam-column element in the two programs: fiber-section in OpenSees and moment-curvature relationship translated to moment-rotation relationship in CAPP.

Figure 7.4. Comparison of pushover curves in longitudinal direction of the Imperial County Services Building from analytical models in OpenSees, CAPP and CANNY.
CHAPTER 8. PUSHOVER CURVES

Pushover curves for the selected building in the transverse (North-South) and longitudinal (East-West) directions were developed for the fundamental-mode height-wise distribution of lateral loads. The multi-linear idealization of the pushover curve was developed from the procedure specified in the FEMA-356 and ASCE-41 documents. Based on the elastic stiffness, $K_i$, and effective stiffness, $K_e$, defined as the initial elastic slope of the pushover curve and initial elastic slope of the bilinear idealization, the “effective” period, $T_e$, was computed from Equation 6.2. Also computed are the base-shear strength as a fraction of the total building weight, and the peak roof (or target node) displacement, $u_t$, recorded during the selected earthquake. The material properties used in developing these pushover curves are the nominal properties specified in the structural drawings.

**Imperial County Services Building**

The pushover curve in the longitudinal direction shows that the Imperial County Services Building begins to rapidly loose strength in the longitudinal direction at roof displacement of about 13 cm (Figure 8.1a). This rapid loss of strength is an indication of initiation of failure (or collapse) of the building. The strong-motion data from the 1979 Imperial Valley earthquake indicated a peak roof displacement in the longitudinal direction of 23.58 cm, which far exceeded the displacement capacity of the building in this direction. As a result, the building is expected to collapse during the selected earthquake, an observation which is consistent with the field report (ATC-9, 1984) that collapse in this building initiated primarily due to motions in the longitudinal direction. The pushover curve in the transverse direction, however, does not indicate collapse as the building’s displacement capacity exceeded the displacement demand of 5.57 cm (Figure 8.1b).

It must be noted that the failure of the building in the longitudinal direction could only be predicted by considering concrete model with crushing in compression. Pushover analysis of
analytical models in *OpenSees* or *CANNY* (see pushover curve in Figure 7.4), which did not consider a concrete model with complete loss of strength immediately after crushing, did not predict the building failure prior to the peak roof displacement.

![Pushover curves for the Imperial County Services Building](image)

**Figure 8.1.** Pushover curves for the Imperial County Services Building.

**Sherman Oaks Commercial Building**

The pushover curve of the Sherman Oaks building in the longitudinal direction indicates that the building was deformed beyond the elastic limit during the 1994 Northridge earthquake: the peak roof displacement of 33.6 cm is slightly larger than the effective yield displacement of about 20 cm (Figure 8.2a). The pushover curve, however, suggests that the building would have collapsed if the roof displacement in the longitudinal direction were to exceed approximately 45 cm due to initiation of rapid loss of strength after this value of roof displacement. The pushover curve in the transverse direction indicates that the building essentially remained elastic in this direction during the 1994 Northridge earthquake as the peak roof displacement is slightly lower than the effective yield displacement (Figure 8.2b).
In addition to the pushover curves for the entire building (Figure 8.2), it is also useful to examine the force-deformation behavior of individual frames. Such results presented in Figure 8.3 for the Sherman Oaks building indicate that the strength of interior frame is significantly larger than that of the exterior frame: exterior frame is about 2.5 times stronger in the longitudinal direction and about 2.0 times stronger in the transverse direction compared to the interior frame. More importantly, the interior frame remains essentially elastic during the 1994 Northridge earthquake, whereas the exterior frame experienced significant nonlinear action. It must be noted that the Sherman Oaks building suffered significant cracks at many beam-column joints (Shakal et al., 1994). The pushover curves, in particular, in the longitudinal direction clearly indicate the possibility of such damage.
North Hollywood Hotel

The pushover curves for the North Hollywood Hotel indicate that the building remained well within the linear elastic range both in the longitudinal as well as transverse direction during the 1994 Northridge earthquake (Figure 8.4). This building is reported to have suffered heavy nonstructural and content damage but no significant structural damage (Naeim, 1999). The lack of structural damage is consistent with the observations from pushover curves in Figure 8.4.

![Pushover Curves for North Hollywood Hotel](image)

Figure 8.4. Pushover curves for the North Hollywood Hotel.

Watsonville Commercial Building

The pushover curves for the Watsonville building indicates that the strength of the building in the longitudinal direction is much lower compared to that in the transverse direction: the building strength is about 0.159W in the longitudinal direction compared to 0.293W in the transverse direction (Figure 8.5). Such is the case because the south face of the building has essentially open first story as opposed to shear walls on the remaining three faces. Furthermore, the building was deformed slightly beyond the elastic range in the longitudinal (or East-West) direction but remained essentially elastic in the transverse (or North-South) direction during the 1989 Loma Prieta earthquake.
Santa Barbara Office Building

The pushover curves for the Santa Barbara building indicate significant strength of the building compared to what may be expected in typical buildings designed in California: the building strength is 0.588W and 0.456W in the longitudinal and transverse directions, respectively (Figure 8.6). Such higher strengths are due to strengthening of the building with large number of shear walls in both directions in 1975. This building remains well within the linear elastic limit in the longitudinal direction but reaches just about the effective elastic limit in the transverse direction during the 1978 Santa Barbara earthquake.

Figure 8.5. Pushover curves the Watsonville Commercial Building.

Figure 8.6. Pushover curves for the Santa Barbara Office Building.
CHAPTER 9. EVALUATION OF NONLINEAR STATIC PROCEDURES

Current nonlinear static procedures (NSPs) are evaluated by comparing the estimated of seismic demands from FEMA-356 CM, ASCE-41 CM, ATC-40 CSM, and FEMA-440 CSM with the values derived from recorded motions of the five selected buildings. It must be noted that the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM, and FEMA-440 CSM are typically restricted to buildings that respond primarily in the fundamental mode. In this investigation, however, these procedures were applied to buildings that may have significant contributions from higher modes, e.g., Imperial County Services Building, Sherman Oaks Commercial Building, and North Hollywood Hotel. The results are presented in this chapter for peak roof displacement and height-wise distribution of various response quantities.

Estimation of Peak Roof Displacement

The application of the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM, and FEMA-440 CSM to estimate the peak roof displacement is illustrated in this section for each of the five selected buildings. The peak roof displacement in the FEMA-356 CM was computed from Equations 6.1 to 6.5 with the coefficient $C_0$ assumed to be equal to the first-mode participation factor and $C_2$ assumed to be 1.0 for framing type 2 defined in the FEMA-356 document. The $S_a$ needed in Equations 6.1 and 6.5 was computed from the 5%-damped elastic response spectrum of the acceleration recorded at the base of the building in the appropriate direction at vibration period $T_e$. The peak roof displacement in the ASCE-41 CM was similarly computed from Equations 6.8 to 6.10.

The peak roof displacement in the ATC-40 CSM is computed from Equation 6.6 with $C_0$ assumed to be equal to the first-mode participation factor and $S_d$ computed from damped elastic response spectrum of the acceleration recorded at the base of the building in the appropriate direction. Because computation of $T_{eq}$ and $\zeta_{eq}$ in Equation 6.7 needs displacement ductility factor, $\mu$, of the equivalent SDF system, the estimation of the target displacement in the ATC-40 CSM requires an iterative procedure. Although ATC-40 document specified three different procedures, the graphical ATC-40 Procedure is used in this investigation to compute $S_d$. For this purpose, a curve of locus of performance points is developed. Each point on this curve is the pair
of displacement and pseudo-acceleration of an equivalent SDF system with $T_{eq}$ and $\zeta_{eq}$ computed for a selected value of $\mu$. The value of $S_d$ to be used in Equation 6.6 is selected as the displacement at the intersection of the curve of locus of performance points and the force-deformation relationship (or capacity) curve of the equivalent inelastic SDF system of the building. The capacity curve of the of the equivalent inelastic SDF system is obtained from the pushover curve of the building by scaling the roof displacement by $1/(\Gamma_1 \phi_1)$ and base shear by $1/M_1^*; \Gamma_1$ is the first-mode participation factor, $\phi_1$ is the first-mode component at the roof (or target node), and $M_1^*$ is the first-mode effective mass.

The peak roof displacement in the FEMA-440 CSM is computed from Equation 6.13 with $S_d$ estimated from a procedure similar to that described for the ATC-40 CSM with two differences. First, values of $\zeta_{eff}$ and $T_{eff}$ in the FEMA-440 CSM are computed from Equation 6.14. Second, the pseudo-acceleration of an equivalent SDF system is modified by a factor to account for the differences between effective period being used in the FEMA-440 CSM and the secant period used in the ATC-40 CSM. Further details of this procedure, denoted as the modified ADRS procedure, are available in the FEMA-440 document.

Typically, the locus of performance points in the ATC-40 and FEMA-440 CSM is plotted on the capacity curve for the equivalent inelastic SDF system to estimate the displacement $S_d$. In this investigation, the displacement $S_d$ is used to compute peak roof displacement which is then plotted directly on the pushover curve of the building. Such a plot permits direct comparison of target displacement from the CSM procedure and the recorded displacement.

The error in the peak roof displacement, $u_c$, from an NSP, compared to the peak roof displacement, $u_t$, derived from recorded motions, is defined as

$$E = 100 \times \frac{u_c - u_t}{u_t} \quad (9.1)$$

Note that the peak roof (or target node) displacement derived from recorded motions is considered to be the exact value in computing the error.
Figure 9.1 presents the peak roof displacement of the Imperial County Services Building in the transverse direction computed from the four NSPs along with the peak roof displacement derived from recorded motions during the 1979 Imperial Valley earthquake. The NSPs could not be applied to estimate peak displacement of this building in the longitudinal direction because the building did not possess sufficient displacement capacity in this direction to withstand the demands imposed by the earthquake, as apparent from the peak recorded displacement being larger than the displacement capacity of the building in Figure 8.1a.

The presented results indicate that the FEMA-356 CM provides an estimate of the roof displacement that is larger than the recorded value during the 1979 earthquake (Figure 9.1a). The overestimation in the roof displacement by the FEMA-356 CM is by about 20%: the value from the FEMA-356 CM is 6.99 cm compared to the recorded value of 5.78 cm. The ASCE-41 CM provides a larger estimate of the roof displacement (Figure 9.1b) compared to the FEMA-356 CM. The overestimation in the roof displacement by the ASCE-41 CM is about 30% compared to about 20% by the FEMA-356 CM. The differences between the peak roof displacements from the FEMA-356 CM and the ASCE-41 CM are clearly due to different values of the factor that converts the peak displacement of a linear-elastic SDF system to that of an inelastic SDF system between the two CM procedures. Recall that the factor to convert the peak displacement of a linear-elastic SDF system to that of an inelastic SDF system is equal to $C_1C_2C_3$ for the FEMA-356 CM (Equation 6.1) and $C_1C_2$ for the ASCE-41 CM (Equation 6.8). Furthermore, values of these individual coefficients between the two procedures differ for the same value of $R$ and $T_e$ (see Equations 6.3 and 6.4 for FEMA-356 CM, and Equations 6.9 and 6.10 for ASCE-41 CM).

The ATC-40 CSM provides a very good estimate of the peak roof displacement of the Imperial County Services Building in the transverse direction (Figure 9.1c). The estimate from the ATC-40 CSM is within about 2% of the recorded value: value from ATC-40 CSM is 5.64 cm compared to the recorded value of 5.78 cm. The FEMA-440 CSM also provides an estimate of the peak roof displacement that is within about 5% of the recorded value (Figure 9.1d). It must be noted that both the ATC-40 CSM and FEMA-440 CSM provide slightly un-conservative estimate of the peak roof displacement of the Imperial County Services Building.
The results presented for the Imperial County Services Building also indicate that the ASCE-41 CM, which was proposed as an improvement over the FEMA-356 CM, does not provide a better estimate of the peak roof displacement compared to the FEMA-356 CM. As shown in Figure 9.1b, the estimate from the ASCE-41 CM is further away, with an error of approximately 30%, from the recorded roof displacement compared to that value from the FEMA-356 CM, which has an error of about 20%. The peak roof displacements from the two CSM procedures – ATC-40 CSM and FEMA-440 CSM – for this building, however, are very close thus making it difficult to conclude if the FEMA-440 CSM, which was proposed as an improvement over the ATC-40 CSM, provides better or worse estimate compared to the ATC-40 CSM. For this building, the CSM procedures provide better estimate of the peak roof displacement compared to the CM procedures.

![Graphs showing peak roof displacements](image)

Figure 9.1. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the transverse direction of the Imperial County Services Building.
Sherman Oaks Commercial Building

The presented results indicate that the peak roof displacements of the Sherman Oaks building in the longitudinal direction computed from the FEMA-356 CM and ASCE-41 CM are identical: the roof displacement is 28.04 cm (Figure 9.2a and 9.2b). Such is the case because the coefficient $C_1$ in the FEMA-356 CM (Equation 6.3) and $C_1$ and $C_2$ in the ASCE-41 CM (Equations 6.9 and 6.10) are all equal to unity because fundamental longitudinal vibration period of this building is the longer than the threshold period value and $C_3$ in the FEMA-356 CM (Equation 6.4) is equal to unity due to positive post-yield stiffness. The two CM procedures provide an estimate of the peak roof displacement that is lower than the recorded value with the underestimation by about 15%: value from FEMA-356 CM and ASCE-41 CM is 28.04 cm compared to the recorded value of 33.6 cm (Figures 9.2a and 9.2b).
Figure 9.2. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the longitudinal direction of the Sherman Oaks Commercial Building.

The ATC-40 CSM provides peak roof displacement of 24.25 cm which is about 30% lower than the recorded value of 33.6 cm (Figure 9.2c). The FEMA-440 CSM provides a slightly better estimate of 27.05 cm (Figure 9.2d) but this estimate is also about 20% lower than the recorded value. Unlike the two CM procedure, the two CSM procedures lead to slightly different values of the roof displacement. This difference is due to different values of effective period and damping ratio used in these CSM procedures (see Equation 6.7 and 6.14). The estimate of the longitudinal roof displacement of the Sherman Oaks Commercial Building is worse from the two CSM procedures compared to the two CM procedures.

All four NSPs lead to identical peak roof displacement in the transverse direction: the peak roof displacement is equal to 17.98 cm (Figure 9.3). Such is the case because the building in the transverse direction remains in the linear elastic range during the 1994 Northridge earthquake. Recall that the coefficients $C_1$, $C_2$, and $C_3$ in the FEMA-356 NSP (Equations 6.3 and 6.4) as well as the coefficients $C_1$ and $C_2$ in the ASCE-41 NSP (Equations 6.9 and 6.10) are equal to one is a system remains in the linear elastic range, i.e., $R = 1$. Furthermore, the vibration period and damping ratio in the ATC-40 CSM and FEMA-440 CSM remain equal to that of a linear-elastic system for $\mu = 1$ (see Equations 6.7 and 6.14). All four NSPs underestimate the roof displacement in the transverse direction by about 20%.

The presented results also indicate that the peak roof displacements from NSPs for the Sherman Oaks building are less that those from recorded motions. Such is the case because the NSPs attempt to capture the response only due to the fundamental mode. Such procedures, obviously, can not capture the response due to higher modes; several higher modes contribute to the response of the Sherman Oaks Commercial Building (Figure 3.5).
Figure 9.3. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the transverse direction of the Sherman Oaks Commercial Building.

**North Hollywood Hotel**

All four NSPs provide identical estimate of the peak roof displacement in the longitudinal direction (Figure 9.5): the peak roof displacement is 10.17 cm. For reasons similar to those as noted previously for the Sherman Oaks Commercial Building in the transverse direction, this occurs because the North Hollywood Hotel also remained within the linear-elastic range during the 1994 Northridge earthquake. The NSPs provide an estimate of the peak roof displacement that is within 5% of the recorded value: the peak roof displacement of from NSPs is 10.17 cm compared to the recorded value of 9.75 cm. Note that the fundamental-mode based NSPs provide very good estimate of the roof displacement because the longitudinal response of this building is primarily due to the fundamental mode (Figure 3.7).
Figure 9.4. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the longitudinal direction of the North Hollywood Hotel.

Similar to the longitudinal direction, the North Hollywood Hotel remained in the linear-elastic range in the transverse direction as well during the 1994 Northridge earthquake (Figure 9.5). Therefore, all four NSPs provide estimate of peak roof displacement that is identical and equal to 14.33 cm. This estimate is lower by about 20% compared to the recorded value: the estimate from NSPs is 14.33 cm compared to the recorded value of 17.46 cm. As noted previously for the Sherman Oaks Commercial Building, the lower estimate from the NSPs is due to inability of NSPs to capture higher mode effects that contribute significantly to the transverse response of this building (Figure 3.8).
Figure 9.5. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the transverse direction of the North Hollywood Hotel.

Watsonville Commercial Building

The presented results for the Watsonville Commercial Building indicate that the FEMA-356 CM provides an estimate of the peak roof displacement that is higher by about 7% compared to the recorded value: the estimated value is 3.56 cm compared to the recorded value of 3.33 cm (Figure 9.6a). The ASCE-42 CM provides estimate of the peak roof displacement that higher by about 40% than the recorded value: the estimated value is 4.79 cm compared to the recorded value of 3.33 cm (Figure 9.6b). The FEMA-356 CM provides an estimate of the peak roof displacement that is much better compared to the ASCE-41 CM. As noted previously for the Imperial County Services Building in the transverse direction, the two CM procedures provide different estimates of peak roof displacement because various coefficients in these procedures
differ for shorter vibration period such as that of the Watsonville Commercial Building; the fundamental longitudinal vibration period of this building is 0.313 sec (Figure 5.6a).

Figure 9.6. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the longitudinal direction of the Watsonville Commercial Building.

The curve of locus of performance points for the ATC-40 CSM and FEMA-440 CSM did not intersect the capacity curve of the Watsonville Commercial Building in the longitudinal direction. The failure of the locus curve to intersect the capacity curve may typically be interpreted as potential instability (or collapse) due to lower strength capacity than the strength demand imposed by an earthquake. However, the field observations do not indicate failure of this building. Therefore, the failure of the locus curve to intersect the capacity curve appears to be due to limitations of the CSM procedures. Note that several previous investigations have pointed
to the convergence failure in the iterative CSM procedures (e.g., Miranda and Akkar, 2002; Chopra and Goel, 2000).

The estimate of the peak roof displacement of the Watsonville Commercial Building in the longitudinal direction from the two CSM was computed by assuming the displacement ductility factor, $\mu$, needed in estimating the effective vibration period and damping ratio (Equations 6.7 and 6.14) to be equal to the peak recorded roof displacement divided by the yield displacement from the pushover curve. The presented results indicate that the ATC-40 CSM provides an estimate of the peak roof displacement that is below the recorded roof displacement by about 40%: the estimated value is 2.04 cm compared to the recorded value of 3.33 cm (Figure 9.6c). The FEMA-440 CSM provides an estimate of the peak roof displacement that is much closer, within about 8%, to the recorded value: the estimate is 3.05 cm compared to the recorded value of 3.33 cm (Figure 9.6d). The FEMA-440 CSM provides a much better estimate of the peak roof displacement compared to the ATC-40 CSM for this building.

The FEMA-356 CM provides an estimate of the peak roof displacement in the transverse direction that is below the recorded value by about 10%: the estimated value is 1.73 cm compared to the recorded value of 1.93 cm (Figure 9.7a). The ASCE-41 CM provides an estimate of the peak roof displacement that is within about 15% of the recorded value: the estimated value if 1.62 cm compared to the recorded value of 1.93 cm (Figure 9.7b).

The ATC-40 CSM provides an estimate of the peak roof displacement that is above the recorded value by about 60%: the estimated value is 3.09 cm compared to the recorded value of 1.93 cm (Figure 9.7c). The FEMA-440 CSM provides an estimate of the peak roof displacement that is above the recorded value by about 63%: the estimated value is about 3.14 cm compared to the recorded value of 1.93 cm (Figure 9.7d). None of the two CSM procedures provides an accurate estimate of the peak roof displacement of this building in the transverse direction.
Unlike the peak roof displacements of the Sherman Oaks Commercial Building and the North Hollywood Hotel, the two CM or the two CSM procedures do not provide an identical estimate of the peak roof displacement of the Watsonville Commercial Building in the transverse direction even though the building remains within the linear-elastic range (Figure 9.7). This occurs due to possible discrepancy between (1) the effective fundamental vibration period estimated from Equation 6.2 and the vibration period estimated from the pushover curve, and (2) the actual damping ratio and the damping ratio of 5% of the linear-elastic system assumed in this investigation. Further explanation of this discrepancy is as follows.

Recall that the pushover curve of a building can be converted to the capacity curve of the equivalent inelastic SDF system of the building by scaling the roof displacement by $1/(\Gamma \phi_{1})$.
and base shear by $1/M_1^*; \Gamma_1$ is the first-mode participation factor, $\phi_r$ is the first-mode component at the roof (or target node), and $M_1^*$ is the first-mode effective mass. The initial elastic slope of the capacity curve of the equivalent inelastic SDF system is equal to $\omega_1^2$. Therefore, the effective initial elastic vibration period can be computed from the capacity curve of the equivalent inelastic SDF system.

The capacity curves of the equivalent inelastic SDF system of the Watsonville Commercial Building in the two directions are presented in Figure 9.8. Included in this figure are: (1) vibration period, $T_i$, computed from the eigen analysis of the linear-elastic model of the building; (2) effective vibration period, $T_\text{e}$, computed from Equation 6.2; and (3) vibration period, $T_1$, computed from initial elastic slope of the idealized capacity curve. Recall that the vibration period, $T_i$, was computed by assuming cracked moment of inertia of the wall to be equal to 0.5 times the gross moment of inertia as per the FEMA-356 recommendations. Also recall that the pushover curve of the building utilized fiber-section for modeling shear walls in the Watsonville Commercial Building. The axial load on the shear walls was minimal because of a separate gravity load system in this building.

![Figure 9.8.](image)

The results in Figure 9.8 indicate that the fundamental vibration period $T_1$ computed from capacity curve of the equivalent inelastic SDF system is much longer than $T_i$ computed from
eigen analysis and $T_e$ computed from Equation 6.2 in both directions. The longer vibration period from the capacity curve is indicative of lower initial elastic stiffness of the system during the pushover analysis compared to that in the model used for eigen analysis. The lower stiffness of the system during pushover analysis is apparently due to lower effective moment of inertia of the shear walls compared to the value of 0.5 times the gross moment of inertia assumed in the model for eigen analysis of the building. This observation is supported by a recent study (Elwood et al., 2007) which concluded that the factor to convert the gross moment of inertia to the effective moment of inertia is significantly lower than the value of 0.5 specified in the ASCE-41 and FEMA-356 documents. The discrepancy can be particularly large for low values of axial force; experimental data presented in Elwood et al. (2007) indicated that the factor can be as low as 0.1 for zero axial force level. Clearly, the vibration period $T_i$ computed based on the effective moment of inertia factor of 0.5, the value specified in FEMA-356 document, would be much shorter compared to the value $T_i$ computed from the initial elastic slope of the pushover curve which used a fiber-section model with very low axial force for nonlinear beam-column element.

For linear-elastic systems, it is expected that the two CM procedures provide the same value of the peak roof displacement as was the case for the Sherman Oaks Commercial Building in the transverse direction and the North Hollywood Hotel in both directions (Figures 9.3 to 9.5). For the Watsonville Commercial Building, however, the two CM procedures provide different estimates of the peak roof displacement. This occurs because the shorter period $T_e$ leads to higher strength of the SDF system needed to remain elastic which in turn leads to value of $R$ becoming larger than one. As noted previously for the Imperial County Services Building in the transverse direction, the two CM procedures would lead to different estimate of the roof displacement because of different values of the coefficients for short period systems; note that the $T_e = 0.279$ sec for the Watsonville Commercial Building in the transverse direction.

For linear-elastic systems, it is also expected that the performance point computed for vibration period and damping ratio of the linear elastic system, i.e., displacement ductility equal to unity, in the two CSM procedures fall on the linear-elastic portion of the pushover curve. This was indeed the case for the Sherman Oaks Commercial Building in the transverse direction and the North Hollywood Hotel in the longitudinal and transverse direction (see Figures 9.3 to 9.5).
For the Watsonville Commercial Building, however, such is not the case. In fact, the performance point corresponding to the vibration period of the linear elastic system is much higher than the linear elastic portion of the pushover curve (see Figures 9.7c and 9.7d). Such is the case because the period of $T_e=0.279$ sec used in the two CSM procedures is much shorter than the actual vibration period (0.503 sec) of the inelastic SDF system (Figure 9.8b). The shorter period leads to much higher spectral acceleration which in turn would lead to much higher base shear demand in the linear elastic system.

The actual intersection of the locus curve and the pushover curve in Figure 9.7 occurs for ductility larger than unity even though the system remains in the linear-elastic range of the pushover curve. This implies that the strength demand of the equivalent linear-elastic system used in the two CSM procedures is larger than that of the actual equivalent elastic SDF system. The higher strength demand is due to two reasons. First, the period, $T_e$, used in the two CSM procedures is shorter than the actual period, $T_i$; the spectral acceleration is typically larger for shorter periods. Second, the damping ratio of 5% used in estimating the peak demand of the equivalent elastic SDF system may be much smaller than the actual value; the apparent damping ratio for shear wall buildings may be higher due to dissipation of energy resulting from soil-structure interaction. Although results are not presented here for reasons of brevity, use of vibration period equal to 0.503 sec, i.e., the value based on the pushover curve, and a damping ratio slightly higher than 10% led to intersection of the locus curve and the linear-elastic portion of the pushover curve at displacement ductility of one.

**Santa Barbara Office Building**

The results presented for the Santa Barbara Office Building indicate that the FEMA-356 CM leads to estimate of the roof displacement in the longitudinal direction that is about 45% lower than the recorded value: the estimate from FEMA 356 CM is 0.36 cm compared to the recorded value of 0.68 cm (Figure 9.9a). The ASCE-41 CM leads to the peak roof displacement that is about 50% below the recorded value: the estimate from the ASCE-41 CM is 0.35 cm compared to the recorded value of 0.68 cm (Figure 9.9b). Note that the two CM provided slightly different values even though the system remains in the linear elastic range. Such is the case
because the compute value of $R$ is larger than unity due to discrepancy in the fundamental vibration period similar to that noted for the Watsonville Commercial Building.

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Figure 9.9. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the longitudinal direction of the Santa Barbara Office Building.

The ATC-40 CSM provides an estimate of the peak roof displacement that is lower than the recorded value by about 15%: the estimated value from the ATC-40 CSM is 0.57 cm compared to the recorded value of 0.68 cm (Figure 9.9c). The FEMA-440 CSM provides an estimate that is lower by about 17% compared to the recorded value: the estimated value from the FEMA-40 CSM is 0.56 cm compared to the recorded value of 0.68 cm (Figure 9.9d). For reasons similar to those noted previously for the Watsonville Commercial Building, the two CSM procedures did not provide identical estimate of the peak roof displacement even though the building remains in the linear elastic range during the 1989 Loma Prieta earthquake.
Figure 9.10. Computation of the roof displacement from the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM and FEMA-440 CSM in the transverse direction of the Santa Barbara Office Building.

The results in the transverse direction (Figure 9.10) also lead to trends that are similar to the ones noted for the longitudinal direction of the Santa Barbara Commercial Building. In particular, the two CM procedures do not lead to identical estimates of peak roof displacement as is expected for buildings remaining in the linear-elastic range. Similarly, the two CSM procedures also do not lead to identical estimate of the peak roof displacement as is expected for buildings remaining in the linear-elastic range. The peak roof displacement from FEMA-356 CM is about 17% below, from ASCE-41 CM is about 30% below, ATC-40 CSM is 17% below and FEMA-440 CSM is about 7% below the recorded value.

**Error in Peak Roof Displacement**

Figure 9.11 shows the percent error (see Equation 9.1) in the peak roof displacement
from various NSPs when compared to the value derived from recorded motions. The results are presented for Imperial County Services Building in the transverse direction (IC-NS), Sherman Oaks Commercial Building in the longitudinal and transverse directions (SO-EW and SO-NS), North Hollywood Hotel in the longitudinal and transverse directions (NH-EW and NH-NS), Watsonville Commercial Building in the longitudinal and transverse directions (WT-EW and WT-NS), and Santa Barbara Office Building in the longitudinal and transverse directions (SB-EW and SB-NS). These results indicate significant errors in the estimate of peak roof displacement from current NSPs. The errors range from about 50% underestimation, e.g., as is the case for FEMA-356 CM and ASCE-41 CM for the Santa Barbara Office Building in the longitudinal direction (see SB-EW in Figure 9.11), to about 60% overestimation, e.g., ATC-40 CSM and FEMA-440 CSM for the Watsonville Commercial Building in the transverse direction (see WT-NS in Figure 9.11).

Among the two CM procedures, the ASCE-41 CM, which is based on the improvements suggested recently in the FEMA-440 document, does not necessarily provide improved estimates for the selected buildings. For example, the ASCE-41 CM leads to larger overestimation for the Imperial County Services Building (see IC-NS in Figure 9.11) and larger underestimation for the Santa Barbara Office Building (see SB-EW and SB-NS in Figure 9.11) when compared to the results from the FEMA-356 CM.

The FEMA-440 CSM generally provides better estimated of the peak roof displacement compared to the ATC-40 CSM several buildings (see SO-EW, WT-EW, and SB-NS in Figure 9.11). For a few other buildings, the FEMA-440 CSM provides estimate that is only slightly worse compared to the ATC-40 CSM (see WT-NS and SB-EW in Figure 9.11). This indicates that the improvements to the CSM procedure suggested in the FEMA-440 document are likely to
lead to better estimated of peak roof displacement.

Finally, there is no clear evidence of whether the CM procedure (FEMA-356 or ASCE-41) or the CSM procedure (ATC-40 or FEMA-440) provides better estimate of peak roof displacement when compared with the value derived from recorded motions. The CSM procedure lead to better estimates for some building (see IC-NS and SB-EW in Figure 9.11) but worse estimates for other (see SO-EW and WT-NS in Figure 9.11) compared to the CM procedure. For other buildings, the two procedures lead to essentially similar level of accuracy (see SO-NS, NH-EW, and NH-NS in Figure 9.11).

Figure 9.11. Percent error in peak roof displacements from various NSPs.
Height-Wise Distribution of Floor Displacements and Story Drifts

Height-wise distribution of peak floor displacements and story drifts from the NSP is compared with that from interpolation of recorded motions for the five selected buildings in Figure 9.12 to 9.20. As noted in the preceding section, various NSP methods generally lead to different values of the peak roof displacement. In order to eliminate the error associated with estimation of the peak displacement, the height-wise distribution of peak floor displacement and story drifts in the NSP were computed for the value of the peak recorded roof (or target node) displacement.

Imperial County Services Building

The results are presented only for the transverse direction of the Imperial County Services Building as the NSP could not be implemented in the longitudinal direction due to limited displacement capacity of this building in this direction. The presented results indicate that the NSP generally provides very good estimates of floor displacements (Figure 9.12a). However, the NSP tends to overestimate the drift in the first story and underestimate them in upper stories (Figure 9.12b). The underestimation of drifts in upper stories is clearly due to inability of the NSP to capture higher mode effects as apparent from the story drifts estimated from recorded motions (Figure 9.12b).

The height-wise distribution of story drifts from the NSP indicates almost a soft-first story behavior of this building: the drift is concentrated in the first story with essentially identical drifts in all upper stories (Figure 9.12b). This behavior is due to due to single-bay shear walls in the first story and walls along the entire width of the building in upper stories of this building in the transverse direction (see Figure 2.3).
Figure 9.12. Height-wise distribution of floor displacement and story drifts of the Imperial County Services Building in the transverse direction.

Sherman Oaks Commercial Building

The results for the Sherman Oaks Commercial Building indicate that the NSP provides a slight overestimation of floor displacements in the longitudinal direction (Figure 9.13a) but very good estimates in the transverse direction (Figure 9.14a). While the NSP provides reasonable estimate of drift in the lower story, it underestimates drifts in upper stories with the discrepancy being significantly larger in the transverse direction (Figures 9.13b and 9.14b). Such is the case because the NSP does not capture the higher mode effects that are present in this long-period building; recall that the fundamental vibration period of this building exceeds 2 sec in both directions.

The height-wise distribution of story drifts also indicates a soft-story condition in the Sherman Oaks Commercial Building: significantly higher drifts occur in the story between ground floor and 2nd floor of this building in both directions (Figures 9.13b and 9.14b). This is due to much taller height of this story compared to all other stories of this building (see Figure 2.7). The presented results indicate that the NSP is able to provide reasonably accurate estimate of the drift in this soft story.
**North Hollywood Hotel**

The NSP provides a very good estimate of floor displacements in the longitudinal direction of the North Hollywood Hotel (Figure 9.15a). The estimation of the floor displacements in the transverse direction, however, appear to be poor (Figure 9.16a). In particular, the NSP provides lower estimates in the lower few floors, higher estimates in floors 9 to 20, and lower estimates in the remaining upper floors. The displacement profile from recorded motions indicates presence of higher modes in the transverse direction (Figure 9.16a). Clearly, the NSP does not capture these higher mode effects and thus provides a poor estimate of floor...
displacements. Note that the target node for this building was selected at the 20th floor and not at the roof. For this reason, the floor displacement from the NSP match the recorded displacement at the 20th floor and not at the roof in Figures 9.15a and 9.16a.

As noted previously for the Sherman Oaks Commercial Building, the NSP provides reasonable drifts in lower few stories but much lower drifts in upper stories (Figures 9.15b and 9.16b). The discrepancy is much larger in the transverse direction (Figure 9.16b) compared to the longitudinal direction (Figure 9.15b). This discrepancy is due to inability of the NSP to capture higher mode effects.

The North Hollywood Hotel also exhibits soft-story condition as apparent from large drift in the story between 2nd and 3rd floors (Figures 9.15b and 9.16b). This condition occurs due to taller story height between these floors. The NSP is able to provide reasonable estimate of the drift in the soft story.

![Figure 9.15](image-url)  
Figure 9.15. Height-wise distribution of floor displacement and story drifts of the North Hollywood Hotel in the longitudinal direction.
Figure 9.16. Height-wise distribution of floor displacement and story drifts of the North Hollywood Hotel in the transverse direction.

**Watsonville Commercial Building**

The NSP provides a very good estimate of floor displacements in the longitudinal direction of the Watsonville Commercial Building (Figure 9.17a). In the transverse direction, however, the NSP underestimates displacements at floors 2 to 4 (Figure 9.18a). The NSP provides reasonable estimate of drift in the first story but underestimates drifts in upper stories in the longitudinal direction (Figure 9.17b). In the transverse direction, however, the opposite trend occurs; the NSP underestimates the drift in the first story but provides very good estimates in the upper stories (Figure 9.18b).

The Watsonville Commercial Building also exhibits soft story condition in the longitudinal direction as apparent from large drift in this story (Figure 9.17a). Such a condition occurs due to open first story on the south face of this building (see Figure 2.13). As noted previously, the NSP provides reasonable estimate of the drift in this soft story.
Figure 9.17. Height-wise distribution of floor displacement and story drifts of the Watsonville Commercial Building in the longitudinal direction.

Figure 9.18. Height-wise distribution of floor displacement and story drifts of the Watsonville Commercial Building in the transverse direction.

Santa Barbara Office Building

The results presented in Figures 9.19 and 9.20 indicate that the NSP generally provides reasonable estimates of floor displacements and story drifts of the Santa Barbara office Building. This is to be expected because the NSP is specifically applicable to short-period buildings such as the Santa Barbara Office Building.
Height-Wise Distribution of Story Shears and Story Overturning Moments

Height-wise distribution of peak story shears and story overturning moments from the NSP is compared with that from interpolation of recorded motions for the five selected buildings in Figure 9.21 to 9.29. As mentioned in the preceding section, these height-wise distributions in the NSP were computed at the value of the peak recorded roof (or target node) displacement. For this purpose, the lateral forces needed to push the structure to the peak recorded roof displacement were computed. These forces were then used to compute the story shears and story overturning moment corresponding to the NSP. Alternatively, the story shears and story...
overturning moment could also be computed from equilibrium of column forces in each story during the pushover analysis when the building’s roof displacement becomes equal to the recorded peak roof displacement.

In order to compute the peak story shears and story overturning moments from recorded motions, total accelerations at non-instrumented floors were computed from the interpolation procedure described in Chapter 4. The force at a floor (or inertial force) at each time instant was computed then as floor mass times the floor’s total acceleration at that time instant. The shear and overturning moment at a story at each time instant were computed as the sum of floor forces and sum of moments due to floor forces, respectively, above that story at that time instant. The peak values of shear or overturning moment at a story were computed as the absolute largest value over all time instances.

It is useful to emphasize that story shears and story overturning moments should be computed from equivalent lateral floor forces given by \( f_s = ku \) in which \( k \) is the building’s stiffness matrix and \( u \) is a vector of displacements at its structural degrees-of-freedom. Computation of the story shears and story overturning moments from the floor inertial forces, \( f_I = m\ddot{u} \), neglects the forces generated by structural damping. For low values of damping, use of \( f_I \) instead of \( f_s \) may only lead to small discrepancy. However, the discrepancy can be large if significant damping forces are present.

Also included in Figures 9.21 to 9.29 are the building’s base-shear strengths. Two values of the base-shear strength are included: the lower bound and the upper bound. The lower bound strength is selected as the largest base shear from pushover curve of the building presented in Chapter 8. Recall that these pushover curves were generated for nominal material strength, i.e., the material strength specified on structural drawings. The upper bound strength is computed by scaling the lower bound strength by a factor of 1.67 to account for the larger value of expected material strengths compared to the nominal values and increased material strengths due to strain-rate effects. As mentioned previously in Chapter 7, FEMA-356 document permits estimation of the expected strength by increasing the nominal strength by a factor of 1.5 for concrete and 1.25 for steel. It was also shown based on pushover curve for the Imperial County Services Building (Figure 7.2) that the base-shear strength increases primarily due to increase in the steel strength.
and the percentage increase in base-shear strength is about the same as the increase in the steel strength. Therefore, a factor of 1.25 was used to account for increased values of expected material strengths. Although not stated explicitly in the FEMA-356 document, an increase in strength of about 33% is widely believed to be a reasonable estimate to account for strain-rate effects. Therefore, a factor of 1.33 was used for this purpose.

*Imperial County Services Building*

The height-wise distribution of story shears and story-overturning moments in the transverse direction of the Imperial County Services building is presented in Figure 9.21. As noted previously, the results in the longitudinal direction of this building are not presented because of its failure in this direction. The presented results (Figure 9.21) indicate that the NSP provides estimates of shears and overturning moments in upper stories that are close to those based on recorded motions. In lower stories, however, the NSP provides a much lower values of shears and overturning moments compared to those from recorded motions. The lower estimates from the NSP is to be expected because the NSP attempts to capture demands only due to the fundamental mode whereas higher modes are expected to contribute significantly to demands like story shears and story overturning moments (see Chopra, 2007).

The base-shear demand estimated from NSP is slightly lower whereas from recorded motions is much higher than the lower-bound base-shear strength (Figure 9.21). However, the base-shear demand from recorded motions did not exceed the upper-bound demand. This indicates that the actual strength of the building was much higher than (and closer to the upper bound value) that computed based on nominal material properties and without consideration of strain-rate effects.
Figure 9.21. Height-wise distribution of story shears and story overturning moments of the Imperial County Services Building in the transverse direction.

**Sherman Oaks Commercial Building**

The results in the longitudinal direction indicate that the NSP underestimates the story shears in the most of the upper and middle stories but overestimates them in lower few stories compared to the values from recorded motions (Figure 9.22a). The story overturning moments are underestimated throughout the building height (Figure 9.22b). As noted previously, this underestimation due to NSP is because of higher mode effects that are not captured by the NSP; recall that the fundamental longitudinal vibration period of the Sherman Oaks Commercial Building is 2.67 sec (Figure 5.3) and higher mode effects are expected to be significant for such buildings.

The base shear demand, both from NSP and recorded motions, is less than the lower-bound value (Figure 9.22a). Note that these demands are still higher than the yield value of $0.088W$ noted previously in Figure 8.2a. The demands from NSP and recorded motions are smaller than the upper-bound strength for obvious reasons.
The story shears and story overturning moments in the transverse direction of the Sherman Oaks Commercial Building are significantly underestimated by the NSP compared to the values from recorded motions (Figure 9.23). This large underestimation in demands from the NSP appears, again, to be because of failure of NSP to capture higher mode effects. This becomes apparent by re-examining the floor accelerations of this building in the transverse direction (see Figure 4.4). The floor accelerations indicate a pattern that may be expected due to significant participation of second and third modes: larger acceleration in lower stories, lower acceleration in lower-middle stories, higher accelerations in upper-middle stories, and lower accelerations in upper stories (Figure 4.4). The significant participation of higher modes in the transverse response of the Sherman Oaks Commercial Building is also apparent from the strong peaks in its transfer function at second and third modes (Figure 3.5).
Figure 9.23. Height-wise distribution of story shears and story overturning moments of the Sherman Oaks Commercial Building in the transverse direction.

The base-shear demand from the NSP is much lower than the lower-bound strength (Figure 9.23a). This is to be expected because the pushover curve presented in Figures 8.2b indicated that the Sherman Oaks Commercial Building remained essentially within the linear-elastic range in the transverse direction. The base-shear demand from recorded motions, however, exceeds the lower-bound strength and approached almost the upper-bound strength. The discrepancy between the base-shear demands from the NSP and the recorded motions appears to be again due to higher mode effects; it appears that the building was deformed beyond the linear-elastic range in the second and possibly third modes whereas it remained elastic in the fundamental mode.

**North Hollywood Hotel**

The results for the North Hollywood Hotel indicate that the story shears and story overturning moments from the NSP are much lower compared to the values from recorded motions (Figures 9.24 and 9.25). As noted previously for the Sherman Oaks Commercial Building, this occurs due to higher modes effects not being accounted for in the NSP. Recall that the fundamental vibration periods of the North Hollywood Hotel are 2.39 sec and 2.8 sec in the longitudinal and transverse direction, respectively (see Figure 5.4) and higher mode effects may be expected to be significant for such a building. The discrepancy between the estimates from the NSP and recorded motions is larger in the transverse direction due to possibility of larger
participation from higher modes as apparent from floor accelerations in Figure 4.6 and transfer function in Figure 3.8.

Figure 9.24. Height-wise distribution of story shears and story overturning moments of the North Hollywood Hotel in the longitudinal direction.

The base-shear demand from both the NSP and recorded motions is much lower than the lower-bound strength in the longitudinal direction (Figure 9.24a). In the transverse direction, while the base-shear demand from the NSP remains much lower than the lower-bound strength, base-shear demand from recorded motions approaches the lower-bound strength (Figure 9.25a). This occurs due to reasons similar to those described previously for the Sherman Oaks Commercial Building.

Figure 9.25. Height-wise distribution of story shears and story overturning moments of the North Hollywood Hotel in the transverse direction.
**Watsonville Commercial Building**

The results presented for the Watsonville Commercial Building indicate that the NSP provides significantly lower estimates of story shears and story overturning moments in the longitudinal direction (Figure 9.26). The high story shears and story overturning moments from recorded motions are due to very high floor accelerations recorded in this direction of the building (Figure 4.7). The inability of the NSP to provide accurate estimate of story shears and story overturning moment appears to be due to its inability to capture higher mode effects.

The base-shear demand from NSP is slightly lower than the lower-bound strength (Figure 9.26a). The base-shear demand from recorded motions, however, exceeds even the upper-bound strength. Recall that the Watsonville Commercial Building has shear-walls for seismic loads and a space-frame for gravity loads consisting of concrete encased steel members. It appears that the gravity-load system may have provided significant additional strength in the longitudinal direction of this building.

![Figure 9.26](image)

Figure 9.26. Height-wise distribution of story shears and story overturning moments of the Watsonville Commercial Building in the longitudinal direction.

As noted previously in the longitudinal direction, the NSP provides much lower story shears and story overturning moments in the transverse direction of the Watsonville Commercial Building compared to the values from recorded motions (Figure 9.27). The base-shear demands from both the NSP and recorded motions, however, are less than the lower-bound strength (Figure 9.27a).
Figure 9.27. Height-wise distribution of story shears and story overturning moments of the Watsonville Commercial Building in the transverse direction.

**Santa Barbara Office Building**

The results presented for the Santa Barbara Office Building indicate that the NSP provides estimates of story shears and story overturning moments that are lower than the values from the recorded motions (Figures 9.28 and 9.29). The base-shear demands from the NSP in both directions are less than the lower-bound strengths (Figures 9.28a and 9.28b). While the base-shear demand from recorded motions is less than the lower bound strength in the longitudinal direction (Figure 9.28a), it slightly exceeds the lower-bound strength in the transverse direction but remains below the upper-bound strength (Figure 9.28b). Note that higher base-shear demand in the transverse direction is due to significantly higher acceleration in this direction of the building (see Figure 4.10).
Figure 9.28. Height-wise distribution of story shears and story overturning moments of the Santa Barbara Office Building in the longitudinal direction.

Figure 9.29. Height-wise distribution of story shears and story overturning moments of the Santa Barbara Office Building in the transverse direction.

**Limitation of NSP**

The results presented in the preceding section indicate that the four NSPs – FEMA-356 CM, ASCE-41 CM, ATC-40 CSM, and FEMA-440 CSM – may lead to either significant underestimation or overestimation of peak roof displacement when compared to the peak roof displacement derived from recoded motions. The error ranges from about 50% underestimation to about 60% overestimation for the five building selected in this investigation.
While the NSP may provide reasonable estimate of height-wise distribution of floor displacements, it provides very poor estimate of drifts in upper stories compared to the values derived from recorded motions even when the building is pushed to the same roof displacement in the NSP as the peak roof displacement derived from recorded motions. Such is the case because NSP is unable to capture higher mode effects that are expected to be more significant for story drifts compared to floor displacements.

The NSPs generally provides very poor estimates of story shears and story overturning moments when compared to the values from recorded motions. The discrepancy is especially large in lower stories. The lower estimates of story shears and story overturning moments from the NSP also occur due to its inability to capture higher mode effects which are also much more significant for story shears and story overturning moments compared to floor displacements.

The NSPs are expected to provide good estimates of seismic demands in low-rise, short-period buildings such as the Santa Barbara Office Building. While the NSPs provide reasonable estimate of floor displacement and story drifts for this building, the estimates for story drifts and story overturning moments are very poor when compared to the values from recorded motions. Therefore, it may be unreliable to estimate story shears and story overturning moments from the NSPs.
CHAPTER 10. CONCLUSIONS

The investigation on evaluation of the FEMA-356 CM, ASCE-41 CM, ATC-40 CSM, and FEMA-440 CSM using strong-motion records of five reinforced-concrete building have led to the following conclusions:

1. The pushover curve for the entire building that is used in implementation of the NSP may not truly reveal the extent of nonlinearity in the building during an earthquake. This may occur for buildings in which strength and stiffness properties of lateral-load resisting elements (such as frames, walls) differ significantly.

2. The various NSPs may lead to either significant overestimation or underestimation of the peak roof displacement.

3. It is expected that various NSPs provide identical estimates of peak roof displacement for buildings responding in the linearly-elastic range during an earthquake. While this expectation is found to be valid for flexible (long-period) buildings, it may not be valid for stiff (short-period) buildings.

4. The ASCE-41 CM, which is based on recent improvements to the FEMA-356 CM suggested in FEMA-440 document, does not necessarily provide better estimate of roof displacement for the buildings considered in this investigation.

5. The improved FEMA-440 CSM generally provides better estimates of peak roof displacements compared to the ATC-40 CSM.

6. There is no conclusive evidence that the CM procedures (FEMA-356 or ASCE-41) lead to better estimates of the peak roof displacement compared to the CSM procedure (ATC-40 or FEMA-440) or vice-versa.

7. The NSP provides reasonable estimate of floor displacements but not for story drifts because of its inability to capture higher mode effects.
8. The NSP provides very poor estimates of story shears and story overturning moments. It appears that estimates of these seismic demands from the NSP are unreliable and should not be used.

It must be emphasized that the NSPs are typically designed to be used with smooth spectrum. Ideally, these procedures must be evaluated using a suite of design spectrum compatible ground motions, a wide range of buildings, and statistical analysis of results. Although, the evaluation of various NSPs in this investigation is conducted based on limited data – five buildings and one set of strong motion records for each building – and this investigation has led to some useful observations, it is still not possible to draw definitive conclusions about all aspects of various NSPs. More definitive conclusions may be drawn as additional data becomes available in future.

A detailed investigation on the effects of modeling assumptions on the pushover curve of the Imperial County Services Building led to the following important observations:

The model based on lumped-plasticity (or concentrated hinges) approach to modeling nonlinearity in the beam column element may provide lower failure displacement of the building compared to the model based on fiber-section approach.

Increase in the concrete strength leads primarily to the increase in initial stiffness of the pushover curve whereas increase in steel strength leads to increase in strength of the pushover curve.

Various computer programs and differences in modeling approaches in these programs may lead to significantly different pushover curves.

Practicing engineers extensively use the pushover analysis for shorter buildings, and many are now beginning to use nonlinear RHA for taller buildings. Therefore, it is useful that we fully understand the effects of such modeling assumptions on response prediction. While the
current study was primarily focused on evaluation of the NSP, further investigation into the effects of various modeling assumptions is in progress; results of such investigation would be reported when completed.

A comparison of base-shear demand from recorded motions with the base-shear strength from pushover analysis indicated that the demand exceeds the strength for several buildings. While a rational explanation has been provided for some of the buildings, such as higher than computed strength due to higher material strength compared to nominal values and increased material strength due to strain-rate effects, it may also be useful to examine the accuracy of the interpolated accelerations at non-instrumented floors. Note that the interpolated accelerations were used to compute floor forces which in turn were used to compute base-shear demand. Any errors in computation of accelerations at non-instrumented floors would obviously result in erroneous estimate of base-shear demand from recorded motions.
REFERENCES


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