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FUNDAMENTAL PERIOD OF VIBRATION FOR REINFORCED CONCRETE BUILDINGS

by

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Abstract

The fundamental period of vibration of buildings is one of the parameters that has a significant influence on the earthquake induced lateral forces. The fundamental period of vibration of reinforced concrete buildings measured during major earthquakes are collected and then compared with the empirical values given in the Uniform Building Code (UBC-97) and the National Building Code of Canada (NBCC-95). Comparisons between measured periods and the current code estimations have shown that the code estimation is inadequate for most of the buildings. Although few researches have been conducted, still there is room left to improve the code formulas.

For shear wall buildings, the codes overestimate or underestimate the period of the building considerably. A new formula is proposed in this study to calculate the fundamental period of shear wall buildings. A very good correlation between computed and measured values is obtained. For the moment-resisting frame buildings, the current codes estimation of the fundamental period translate into a conservative values of the design base shear forces. However, when the non-structural elements contribute to the overall stiffness of the building, the code estimation deviates from the measured values substantially. In this research, a study is conducted to evaluate the effect of non-structural elements on the fundamental period of the moment-resisting frame buildings. It is concluded that the fundamental period of the infilled frame is about 25% to 65% of the bare frame's period.

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Notation

A_B	= Plan area of the building
A_c	= Effective area of shear walls
A_e	= Equivalent area of shear walls; Cross-sectional area of the shear wall
$\overline{A_e}$	= Equivalent shear area
A_i	= Plan area of the *i*th shear wall
ATC-06	= Applied Technical Council project (Tentative Provisions)
A_w/A_f	= Wall-area-to-plan-area ratio
B	= Dimension of the building normal to the direction of vibration
CSMIP	= California Strong Motion Instrumentation Program
C_t	= Constant
D	= Dimension of the building parallel to the applied forces
D_e	= Length of the wall in the direction of study
D_i	= Length of the *i*th wall in the considered direction of vibration
D_s	= Length of the main lateral resisting system in the studied direction of vibration
E	= Modulus of elasticity
E_f	= Modulus of elasticity of the frame material
E_m	= Modulus of elasticity of the masonry
EW	= East-West direction of vibration
f'_c	= Compressive strength of concrete
f'_m	= Compressive strength of masonry
G	= Shear modulus
h	= Height of the infill panel
H, h_n	= Total height of the building above the base
H_i	= Height of the *i*th shear wall
I	= Moment of inertia of a section

I_c, I_b	= Moment of inertia of columns and beams, respectively
k	= Stiffness
K	= Numeric coefficient equal to 700 for concrete block and 500 for clay brick
K	= Stiffness matrix
k_I	= Stiffness of the infilled frame
L	= Length of the shear wall or infill panel
L_d	= Length of the diagonal strut, in meters
m	= Mass
M	= Diagonal mass matrix
m_I	= Mass of the infilled frame
MRF	= Moment-resisting frame
N	= Number of storeys above the ground
NBCC	= National Building Code of Canada
NOAA	= National Oceanic and Atmospheric Administration (U.S.)
NS	= North-South direction of vibration
NSMP	= National Strong Motion Program (U.S.)
NW	= Number of shear walls
R	= Wall-area-to-floor-area
R^2	= Correlation factor
SEAOC	= Structural Engineers Association of California (Recommended Lateral Force Requirements)
SW	= Shear Wall
t	= Wall thickness
T	= Fundamental period of vibration of the structure in the direction under consideration
T_I	= Fundamental period of vibration of the infilled frame
UBC	= Uniform Building Code
USGS	= United States Geological Survey
w	= Effective width of the equivalent diagonal strut
α_h	= Length of contact between an infill wall and the columns

α_L	= Length of contact between an infill wall and the beams
λ	= Ratio of the fundamental period of the infilled frame to that of the bare frame for moment-resisting frame buildings
θ	= Angle between the equivalent compression strut and the horizontal
μ	= Poisson's ratio equal to 0.2 for concrete
Φ	= Matrix of eigenvectors or mode shapes
ω	= Natural circular frequency
Ω^2	= Diagonal matrix of eigenvalues

CHAPTER 1

Introduction

1.1 General

Many researchers have studied the effects of earthquakes on building structures. Structural stiffness and mass govern seismic induced inertia forces. The distribution of seismic lateral loads and their magnitude depend on the fundamental period of structure. Therefore, accurate determination of fundamental period results in a realistic level of earthquake forces. However, accurate computation of period is not an easy task at the design stage.

Lately, many buildings have been instrumented to measure their behaviour during earthquakes. The readings obtained during past earthquakes have been of great importance, shedding light on dynamic properties of buildings. The database obtained during seismic activity is enriched after every major earthquake. It is important to correlate the fundamental periods of vibration of existing buildings that have been subjected to strong ground motions, to those of new buildings that are being designed.

Building codes provide some empirical equations that can be used to estimate the period of vibration, based on the geometric dimensions of buildings. These equations are applicable to buildings with different lateral load resisting systems, such as shear walls and moment-resisting frames. They were calibrated and verified using data obtained from instrumented buildings, and have been revised and improved, as more data have become available. Further improvements are needed based on the new data that have been recorded during recent earthquakes.

1.2 Objectives and Scope

The main objective of this research project is to evaluate the empirical equations provided in current building codes for the calculation of fundamental periods of buildings. and recommend possible improvements. The objective also includes the investigation of parameters that affect the fundamental period, while also verifying the analysis technique used against measured data.

The scope is limited to reinforced concrete buildings, and includes moment-resistant frame and shear wall buildings. The following outlines the scope:

- Review of state of the art and previous research.
- Collection of measured periods of instrumented buildings, including those recorded during recent earthquakes.
- Evaluation of current code formulas.
- Investigations of the effects of parameters that affect the period of frame and shear wall buildings.
- Development of improved expressions for the computation of fundamental period of frame and shear wall buildings, suitable for codification.
- Presentation of research findings.

1.3 Previous Research

1.3.1 Empirical Formulas in Building Codes

Over the past three decades, research has been conducted to verify and improve the empirical formulas used for design, in estimating the fundamental period for buildings. These formulas were evaluated using available periods measured during past earthquakes. Generally, the code formulas were calibrated to give shorter periods that those measured, to produce conservative design seismic forces. However, sometimes the degree of conservatism may be excessive, while at other times the period may be overestimated due to the omission of non-structural elements in period calculations, leading to non-conservative design forces. Although some improvements have been introduced over the years, these equations still need to be verified and improved, as new data become available after earthquakes. This is especially true for shear wall buildings for which the period is poorly estimated by code expressions.

Housner and Brady [19] performed some theoretical analyses in 1963 for idealised shear wall buildings to derive a formula to calculate the fundamental period based on Rayleigh's method. They compared the resulting expressions with those suggested by the California Building Code of 1960, and with the period of vibration of 77 steel and reinforced concrete buildings measured during the 1933 Long Beach, California earthquake. They found that the proposed equation, $T= CNB^{1/2}$, gave better results than the code equation, $T= 0.05H/D^{1/2} = CN/D^{1/2}$. It was also shown that the equation suggested by the code for moment resisting frames, $T=0.1N=CN$, gave better estimations than that for shear wall buildings. In these formulas, T is the fundamental period in seconds, H is the height of the building in feet, C is a constant, N is the number of stories, and D and B are the dimensions of the building parallel and normal to the considered direction of vibration, in feet. The researchers concluded that good estimates for shear-wall buildings could be obtained only if the actual wall stiffness was considered.

The analysis of 18 moment-resisting frame buildings was performed by Bendimerad et al. [4] in 1991. The fundamental period formula in the Uniform Building Code of 1988 (UBC-88), $T=C_t H^{3/4}$, where H is the height of the building in feet, was compared to the measured periods of selected buildings. It was concluded that; i) the value of Ct=0.035 represented a lower limit for reinforced concrete moment-resisting frame buildings, and therefore was recommended for use in design, ii) there was no difference between the fundamental period of ductile and non-ductile moment frame buildings. The stiffness of non-structural components was found to be small when compared to the stiffness of the frame. It had no effect on the building period after the first five seconds of earthquake motion. Finally, they suggested that the code formula could be improved by taking into account the relative rigidities in two principal directions of buildings.

Cole et al. [9], in 1992, verified the empirical period formulas in UBC-91 with the data of 64 buildings obtained from their recorded motions during some California earthquakes. It was concluded that, for moment-resisting frame buildings, i) the measured periods were in general longer than the values computed with the code formulas, and ii) the period calculated as 1.4 times the value from the code equation correlated well with the measured data. For shear wall buildings, they concluded that the measured periods were much shorter than the code values, but because of a limit of 2.75 set on the seismic coefficient, it had no effect on base shear calculations. A revision on this limit was also suggested.

Seismic records from the Loma Prieta and Whittier earthquakes for 21 buildings were analysed by Li and Mau [21] in 1997. The measured fundamental period was compared with those specified by the formulas in UBC-94, $T=C_t\ H^{3/4}$. It was observed that for reinforced concrete frame buildings, the fundamental periods were overestimated while for shear wall buildings the periods were largely overestimated or underestimated. It was shown that the values from the alternative code formula for shear wall buildings, $T=0.1/A_c^{1/2}$ (UBC-94), where A_c depended only on the length and height of first story walls, were not better than those generated by previous equations.

Goel and Chopra [15,16] expanded the available database on fundamental period of buildings by adding those measured during the 1971 San Fernando Earthquake. They included data from more recent Californian earthquakes, including those measured during the 1994 Northridge earthquake. The empirical formula $T=C_t H^{3/4}$ for moment resisting frames, where $C_t=0.030$ for reinforced concrete buildings (UBC-97, ATC3-06, SEAOC-96 and NEHRP-94), was compared with measured periods of 27 buildings. It was found that the measured periods were generally longer than those computed by the code expression, leading to conservative values of base shear. Although the code formula was found to be conservative, they proposed the equation $T=0.016 H^{3/4}$, which gave the best correlation with measured data. It was also recommended that the period computed from a rational analysis should not be larger than 1.4 times the value obtained from the proposed equation.

For shear wall buildings, Goel and Chopra [14,16] compared the fundamental period of vibration of 16 shear wall buildings measured from strong motion records with the code formula. $T= C_t H^{3/4}$ with Ct=0.02 (UBC-97, NEHRP-94 and SEAOC-96). They found that the code formula gave a longer period than the measured period, leading to a non-conservative estimation of design base shear. When the alternative value for C_t was used, $C_t=0.1/A_c^{1/2}$ (UBC-97 and SEAOC-96), where A_c was the combined effective area in square feet, it was found that code values were in general much shorter than the measured periods, and were therefore over-conservative. It was shown that the formula specified in ATC3-06, $T=0.05H/D^{1/2}$, where D was the dimension of the building at its base in the direction under consideration in feet, underestimated the period considerably. A new formula was proposed based on Dunkerley's method:

$$T = 0.0019\frac{H}{\sqrt{Ae}}$$

where

$$Ae = \sum_{i=1}^{NW}\left(\frac{H}{Hi}\right)^2 \frac{Ai}{\left[1+0.83\left(\frac{H}{Hi}\right)^2\right]} \qquad \text{and} \qquad \bar{A}_e=100A_e/A_B$$

A_i, H_i and D_i are the area, height and dimension in the direction considered of the *i*th shear wall, in feet; NW is the number of shear walls; and A_B is the building plan area in square feet. They also recommended that the period computed from a rational analysis should not be larger than 1.4 times the value from the proposed equation.

1.3.2 Measured and analytical fundamental periods

The instrumentation of buildings provides invaluable information, increasing knowledge on actual structural behaviour during earthquakes. The data can be used to verify analytical formulas and/or approaches. A great deal of research has been carried out to determine the vibration properties of buildings from their recorded motions during earthquakes and to evaluate the current analytical modelling with the performance of instrumented buildings.

The Imperial County Services Building was analysed by Pardoen et al. [29]. Four reinforced concrete moment resisting frames formed the lateral resisting system of the building in the East-West (E-W) or longitudinal direction. In the North-South (N-S) or transverse direction, end shear walls were provided at the upper five storeys and four asymmetrically located shear walls at the ground floor. A three dimensional model was created using ETABS software. The floors were assumed rigid in their own plane and the horizontal lateral loads were assumed to act at the floor levels. Uncracked gross sections were used for all elements. The fundamental periods were reported to be 0.44 and 0.67 seconds in the N-S and E-W directions, which were close to those obtained from ambient vibration tests [30], 0.45 and 0.65 seconds in the N-S and E-W directions, respectively. When the averaged cracked section properties were used, the resulting periods were closer to those from strong motion records [42].

Hart et al. [17] in 1989, evaluated the seismic response of a 10-storey reinforced concrete building located in Whittier, California, during the Whittier earthquake. The main lateral force resisting system consisted of a moment resisting frame. The identified fundamental period from the strong motion records was approximately 1.1 seconds. The analytical period was obtained using ETABS computer program. The model was analysed assuming three different degrees of cracking for the elements; gross properties for all sections, cracked beams only, and fully cracked (beams and columns). The best correlation between the measured and analytical response was obtained with the fully cracked model, with a fundamental period of 1.17 seconds.

In 1991 Anderson et al. [2], determined dynamic characteristics of a 30-storey reinforced concrete building, located in Emeryville, California from the motions recorded during the Loma Prieta earthquake. The fundamental periods were identified to be 2.59 seconds in the N-S direction and 2.69 seconds in the E-W direction. The periods from ambient vibration and forced vibration were also reported as 1.71 seconds for both directions during ambient vibration. The corresponding values from forced vibration were 1.68 in the N-S direction and 1.69 in the E-W direction, respectively. Safak and Celebi [33] also evaluated the dynamic properties of this building during the Loma Prieta earthquake in 1992. The fundamental periods determined with system identification techniques were 2.50 seconds in the N-S direction and 1.43 seconds in the E-W direction.

In 1991. Miranda and Bertero [24] evaluated the seismic response of a 10-storey building located in Whittier, California, during the Whittier Narrows earthquake. Moment resisting frames in the longitudinal (N-S) direction and shear walls in the transverse (E-W) direction formed the lateral resisting system of the building. The identified fundamental period from strong motion records, in the longitudinal direction was 1.43 seconds. while it was 0.61 seconds in the transverse direction. From the strong motion records during the Whittier earthquake, the periods were reported to be 0.71 and 0.60 seconds in the longitudinal and transverse directions, respectively. The analysis of the building was performed with SAP90 program, with a model that was calibrated to give the same periods as those from the motion records. Cracked and transformed sections were used for the structural system.

The analytical modelling of the San Bruno–Commercial building was carried out by Phan et al. [31]. The lateral force resistance of the building consisted of moment-resisting frames in the N-S (longitudinal) direction and a heavily reinforced concrete frame in the E-W (transverse) direction. First, the model was analysed considering the base as fixed. The fundamental periods were reported as 0.78 in the N-S and 0.82 seconds in the E-W directions. The model was then analysed considering the base as spring-supported to account for soil effects. The resulting periods were 1.10 and 1.15 seconds, respectively. It was observed that, for this building, the increase in the period between the ambient vibration and the strong motion could be in part attributed to the soil interaction effect.

In 1994, Marshall et al. [22] compared the strong motion records during the Loma Prieta earthquake with the ambient vibration measurements for 5 buildings. The buildings included different structural systems and different materials. It was observed that the periods from ambient vibration were, in all cases, shorter than the period from strong motion records, with the largest differences occurred in concrete buildings. They suggested that the fundamental frequencies obtained from ambient vibration tests should be multiplied by a factor varying between 0.8 and 0.9 to estimate the strong motion frequencies.

Ventura and Schuster [41], in 1996 analysed the dynamic properties of a 30-storey reinforced concrete shear wall building located in Vancouver, B. C. Ambient vibration measurements were taken during various stages of construction. When the structure was finished. the measured periods without architectural elements were 1.83 seconds for the N-S direction and 1.51 seconds for the E-W direction. After the architectural elements were in place, the measured periods were 1.83 and 1.55 seconds for the N-S and E-W direction, respectively. Results revealed a little influence of the architectural components on the vibration of this building. A three dimensional dynamic analysis was performed with ETABS computer program. The design periods were found as 2.31 seconds in the N-S (transverse) direction and 1.77 seconds for the E-W (longitudinal) direction. The corresponding values calculated with the UBC-97 were reported as 1.89 and 1.62 for N-S and E-W directions, respectively. The National Building Code of Canada (NBCC) formulas gave 2.72 and 2.33 seconds, respectively. Based on the correlation between experimental and analytical frequencies and mode shapes, it was concluded that the dynamic behaviour of building was accurately estimated.

In 1997, Naeim [25] reported the dynamic characteristics and seismic response of 17 buildings, located in the Los Angeles area, obtained from their recorded motions during the Northridge earthquake. The buildings included a wide range of structural systems. Information about the inspection of the buildings after the earthquake, levels of damage and structural details are provided. An evaluation of the building performance is also discussed.

In 1997, Anderson and Bertero [1] analysed the seismic performance of the San Bruno, Office Building. The lateral resisting system consisted of five external ductile moment-resisting frames, two external in the N-S (longitudinal) direction and three in the E-W (transverse) direction. The fundamental periods from the Loma Prieta earthquake were 1.05 and 0.85 seconds for the N-S and E-W directions, respectively. Initial periods due to low amplitude accelerations were also identified as 0.57 and 0.71 seconds approximately, for the same directions. It was observed that these initial periods correlated well with ambient vibration test results (0.58 and 0.71 seconds, respectively). A three dimensional model was analysed using SAP90 and ETABS computer programs. When cracking of the elements was not considered, the results were in good agreement with the ambient vibration tests. When the rigidity of the columns was decreased to 50%, the computed periods were 1.03 and 0.89 seconds for the N-S and E-W directions, respectively, which were close to the values obtained from strong motion records (1.05 and 0.85 seconds, respectively).

Analytical and measured dynamic behaviour of eight buildings were compared by De la Llera and Chopra [10] in 1998. The buildings had different structural types, and were subjected to the 1994 Northridge Earthquake. A three dimensional model was prepared using SAT computer program. The masses were assumed lumped at floor levels. Each floor was considered as a rigid diaphragm. Three reinforced concrete buildings were evaluated. For the first one, the Van Nuys Hotel, the lateral force resistance was provided by perimeter reinforced concrete frames. The analysis was conducted considering the additional stiffness and lateral resistance produced by interior columns and the flat slab. The computed fundamental periods were reported as 1.34 and 1.59 seconds for the transverse and longitudinal directions, respectively. The periods identified from the recorded motion of the building were 1.94 and 2.08 seconds in the same directions, respectively. The second building was the Los Angeles Warehouse building, consisting of a ductile peripheral reinforced concrete frame system. From the analytical model, the fundamental periods of vibration were computed as 1.16 seconds in the transverse direction and 1.36 seconds in the longitudinal direction. The identified vibration periods from the recorded motion of the building were 1.47 and 1.61 seconds for the transverse and longitudinal directions, respectively. The third building was the Burbank building. The lateral resistance in this building was provided by precast reinforced concrete shear walls in two directions. The measured periods were reported as 0.69 and 0.47 seconds for the longitudinal and transverse directions, respectively, while the analytical periods were 0.63 and 0.41 seconds, respectively.

1.3.3 Infilled frames

It has been shown that architectural components have insignificant effect on the fundamental period of buildings, but this is not the case when masonry is used as partition walls. Usually this type of non-structural element acts together with the structure, adding some stiffness to the frame and modifying the behaviour of the frame. Many researchers have studied the effect of infill walls on the dynamic behaviour of buildings.

Kligner and Bertero [20] tested a 1/3 scale, moment-resisting infilled frame model to determine its behaviour during earthquakes. It was observed that the infills changed the behaviour of the bare frame to that of a braced frame. They also developed a mathematical model based on the equivalent strut concept. They found that the infills increased the stiffness of the frame in about 5 times. Although the fundamental period was reduced and the inertia forces consequently increased, the general behaviour of bare frame was improved through the use of infills.

Meherabi et al. [23] tested a 6-storey, three bay, reinforced concrete moment-resisting frame, designed according to the provisions of UBC-91. Solid and hollow masonry units were used as infills. In general, the performance of frame was improved. It was shown that the lateral force resistance of a infilled frame was higher than that of bare frame. It was concluded that a proper design of the infills would have beneficial effects on the behaviour of frames.

Elouali et al. [2] studied the effect of infill walls on vibration periods of frames. The infill wall was modelled as an equivalent diagonal strut. From their results, it was concluded that the fundamental period of the infilled frame could be reduced in about 50 to 70% from that of the bare frame.

CHAPTER 2

Fundamental Period of Reinforced Concrete Shear Wall Buildings

2.1 General

For many years, extensive research has been conducted to evaluate the fundamental period expressions proposed for shear wall buildings. It has been recognised that the formulas addressed in the current building codes lead to a relatively poor estimation. For simplicity, these formulas are often based on the wall length and the building height. A comparison of these formulas with periods measured from instrumented structures is presented in this chapter. It is argued that if the stiffness of the walls, instead of the wall length is included in the analysis, more accurate results could be obtained.

In order to investigate the importance of structural parameters that affect dynamic characteristics of shear walls, it is essential to perform a parametric investigation. In this regard, the shear wall layout and dimensions constitute parameters of special interest. These parameters have been analysed in the current study, and the results are presented in this Chapter.

2.2 Instrumented Buildings and Measured Fundamental Period Database

The actual structural response of a building, with as-built conditions, can be substantially different than that computed analytically, based on assumptions. The primary difference comes from the modelling techniques used and the non-structural components that are often ignored in structural analyses. It is therefore important to have data on actual building response, as obtained from instruments placed prior to dynamic excitation. In order to evaluate and develop improved expressions for current building codes it is important to calibrate analytical works against data obtained from strong motion instruments on buildings, recording data during major earthquakes. Buildings for which these records are available, were instrumented by the California Strong Motion Instrumentation Program (CSMIP), the National (U.S.) Strong Motion Program (NSMP), and the National (U.S.) Oceanic and Atmospheric Administration (NOAA), all in the U.S.

A collection of fundamental periods of instrumented buildings is presented in Table 2.1. Included in this table are 26 reinforced concrete shear wall buildings, 4 buildings with masonry shear walls and 3 buildings with shear walls in one direction and concrete moment resistant frames in the other. The table also includes properties of these buildings, such as location, number of stories, dimensions, measured periods and the earthquake records from which the periods were measured. Each building is identified by a label. The letter "C" or "N" in the label indicates buildings instrumented by the CSMIP or NOAA, respectively, followed by the station number. The designation ATC, followed by the building number identifies those buildings taken from the ATC-06 report [3].

Fundamental periods reported by different investigators for the same buildings and earthquakes were also taken into account. A few data were measured directly from the strong motion records of the Loma Prieta and Northridge earthquakes. These records were found in published reports [28,35].

2.3 Evaluation of the Existing Formulas in Current Building Codes

A comparison between measured data and the expressions provided in building codes is presented in this section. The building codes considered include both the National Building Code of Canada (NBCC-95) and the Uniform Building Code (UBC-97). The 1995 issue of the National Building Code of Canada [27] specifies that the fundamental period of structures, except the moment resisting frames, shall be calculated using the following formula:

$$T = 0.09\frac{h_n}{\sqrt{D_S}} \tag{2.1}$$

Where h_n and D_s are the height of the building and the length of the wall or braced frame in the direction parallel to the applied forces, in meters, respectively. If the resisting system does not have a well-defined length, then D should be used instead, where D is the dimension of the building in meters in the direction parallel to the applied forces.

The fundamental period can also be calculated using a rational method such as the Rayleigh's method, but the resulting base shear should not be less than 0.8 times the base shear calculated from the empirical formula given in Eq. 2.1.

The 1997 Uniform Building Code [40] states that the period shall be determined as follows:

$$T = C_t\left(h_n\right)^{3/4} \tag{2.2}$$

Where, C_t is equal to 0.073 when h_n is in meters, or 0.030 when h_n is in feet. Eq. 2.2 also applies to reinforced concrete moment-resisting frames as well as eccentrically braced frames. For all other buildings C_t is equal to 0.0488 in the SI system or 0.02 in the imperial system, respectively. In this formulation, h_n is the height of the building above the base. Alternatively, C_t could be calculated for structures with concrete or masonry shear walls as follows:

$$C_t = 0.0743/\sqrt{A_c} \qquad \text{(SI system)} \qquad (2.3)$$

or

$$C_t = 0.1/\sqrt{A_c} \qquad \text{(Imperial system)}$$

Where, A_c is defined as the effective area of the shear walls in the first story of structure, given by;

$$A_c = \sum A_e\left[0.2 + (D_e/h_n)^2\right] \qquad (2.4)$$

A_e is the horizontal cross-sectional area of the first story wall in square meters or square feet, and D_e is the length of the wall in the direction under consideration in meters or feet. The value of D_e/h_n should not exceed 0.9.

UBC-97 also indicates that the fundamental period can be calculated using a rational analysis, such as Rayleigh's method or Eigen-value solution. In this case, the base shear should not be less than 80% of the base shear calculated by Eq. 2.2. The formula specified in ATC-06 and earlier versions of the UBC is the same as the present formula in the NBCC (Eq. 2.1).

2.3.1 The National Building Code of Canada

The measured periods of buildings presented in Table 2.1 are compared with the values obtained with the NBCC-95 formula. First, the length of the building, D, is used in place of D_s. This is permitted when the length D_s is not clearly identifiable. In Figure 2.2 the parameter $h_n/D^{1/2}$ is plotted against measured periods. Measured periods in both the transverse and longitudinal directions are used.

The comparison given in Figure 2.2 indicates that there is a wide scatter of data. The code formula largely underestimates the periods. About 30% of the measured data are lower than the computed values. It may be argued that the empirical expression given in the code is inadequate. This conforms to the results obtained by other researchers (Goel and Chopra [16], Hart et al. [17]). Previous research has shown that the building length, D, is not a good parameter to estimate the period, because buildings with similar lengths may have very different period values, depending on the rigidity of shear walls [16]. This also agrees with the results obtained in the current investigation.

2.3.1.1 Evaluation of the Length of the Lateral-Load Resisting System (D_s)

The length of the main lateral-load-resisting system, D_s, can be easily identified for some buildings, while it can be very confusing for others. The code is short of providing clear guidelines on the estimation of D_s. Some guidelines on the use D_s were developed as part of the current research program to obtain improved estimates of the fundamental period.

The selection of D_s is a relatively easy task for buildings with single isolated walls. Difficulties arise when the walls are coupled with beams. When properly proportioned, the coupling elements enhance performance of shear walls by improving their dynamic characteristics. If the coupling effect is ignored, the fundamental period is over-estimated while the actual seismic force is underestimated. Researchers have proposed different methods to calculate the stiffness of coupled systems based on the stiffness of coupling elements. However, at the early stages of design, the stiffness contribution of coupling elements is unknown and a fairly simple rule is required to establish this quantity. In this research program, a coupled wall is defined as a system consisting of two walls with a connecting beam where the clear beam span is equal to or less than one half of the shorter wall length. This definition is schematically illustrated in Figure 2.1a. Furthermore, a distinction is made between a wall and a column. When the width-to-thickness ratio of a vertical element (column or wall) is greater than 3.0, then it is treated as a wall, and if the ratio is less than 3.0, the element is considered as a column [13].

Buildings with measured data were examined to identify those with sufficient information available to decide on the dimension D_s. These shear wall buildings are marked in Table 2.1 with " * ". The floor plans of these buildings are reproduced in Appendix A-1. The NBCC formula is evaluated for different definitions of D_s, as defined below:

a. The total length of all shear walls in a given direction, divided by the number of walls.
b. The total length of all shear walls in a given direction, divided by the number of frames that contain these walls, as illustrated in Figure 2.1b (All walls are treated as isolated walls).
c. The largest sum of shear wall lengths in any frame.
d. The total length of all shear walls in a given direction, divided by the number of frames that contain these walls, while considering the coupling effect (if conforms to the coupled wall definition specified earlier), i.e. length of coupled walls includes the coupling beam span.

A comparison of the measured periods and the periods computed based on the NBCC-95 formula, with D_s calculated as defined above, are presented in Tables 2.2 to 2.4 and in Figures 2.3 to 2.5. The results are categorised as buildings with isolated shear walls (Table 2.2), coupled shear walls (Table 2.3), and shear walls located at the central core only (Table 2.4). The buildings were evaluated separately in transverse and longitudinal directions.

For isolated walls, the best estimate of the period was obtained when D_s was calculated as the average of wall lengths per frame. Although a large overestimation of 72% of the period was observed for the Piedmont building (C58334), the computed periods were below the measured values with a variation of about 9% to 38%, producing conservative values of the design lateral forces.

The analysis of coupled wall systems indicated that, when the length of coupled walls was defined as shown in (Fig. 2.1a) and used as D_s, the NBCC expression provided a better estimation of periods than using the summation of lengths of individual walls that make of the coupled wall systems. When some of the walls were isolated and others coupled, the best estimate was obtained with D_s calculated as the average of the wall length per frame.

2.3.2 Uniform Building Code

The comparison between the previously measured periods of buildings (Table 2.1) and those obtained by UBC-97 is depicted in Figure 2.6. The fundamental period, T, is plotted against the building height, h_n, (in meters). It can be seen that most of the buildings have longer periods than those obtained with the code formula. It is clear that the measured data is scattered over a wide range, and can not be estimated accurately by a simple empirical expression that is only a function of the building height.

The alternative formula specified in the UBC-97 (Eqs. 2.3 and 2.4) is compared with the measured periods as shown in Figure 2.7. The value A_c is calculated only for the selected buildings (Table 2.1) for which detailed information about the wall dimensions were available. It was observed that the measured periods were much longer than the values given by the code formula. The results indicate that the refinement introduced by calculating the wall area did not produce any appreciable improvement over the earlier expression, which was only a function of the height of the building.

2.4. Formula Proposed by Goel and Chopra

After showing the inadequacy of the current building code formulas to estimate the fundamental period of shear wall buildings, Goel and Chopra [14,15] proposed a new expression to conservatively estimate periods. This formula is based on the Dunkeley's method of considering shear and flexure deformations in a cantilever. The stiffness and properties of individual walls are taken into account, as follows:

$$T = 0.0063\frac{H}{\sqrt{A_e}} \qquad \text{(SI system)} \qquad (2.5)$$

or

$$T = 0.0019 \frac{H}{\sqrt{\bar{A}e}} \qquad \text{(Imperial system)}$$

Where;

$$Ae = \sum_{i=1}^{NW} \left(\frac{H}{Hi} \right)^2 \frac{Ai}{\left[1 + 0.83 \left(\frac{H}{Hi} \right)^2 \right]} \tag{2.6}$$

and

$$\bar{A}_e = 100 \frac{A_e}{A_B} \tag{2.7}$$

A_i, H_i and D_i in the above expressions represent the area, height and length of the i^{th} shear wall at the base, in the direction being considered, in meters or feet. NW is the number of shear walls, and A_B is the plan area of the building in square meters or square feet. The coefficient 0.0063 (or 0.019) was obtained by regression analysis of the measured data. They recommended that the computed period from a rational analysis should not be greater than 1.4 times the value obtained by their proposed equation. These equations are applicable only for uncoupled shear walls, connected by rigid diaphragms.

The comparison of the measured data and the computed periods with the proposed formula is depicted in Figure 2.8. Except for the three different periods reported for San Bruno building (C58394) in the transverse direction, which are greatly underestimated, the periods computed with this formula provide conservative estimates. Few cases are overestimated by about 3% to 29%, and the rest of the periods are underestimated up to 45%. The results lead to conservative estimates of seismic design forces.

2.5 Analytical Computation of Period for Shear Wall Buildings

Analyses of reinforced concrete shear wall structures were carried out to verify the applicability of rational analysis techniques in establishing the fundamental periods of actual building structures and to establish the significance of parameters affecting their period. Some of the instrumented buildings were analysed using the computer software SAP 2000 for verification. Additional idealised buildings were selected to conduct a parametric investigation. The buildings were first modelled to represent their mass and stiffness, for the purpose of analysis.

2.5.1 Verification of the Analysis Procedure

The validity of the analysis procedure and the applicability of computer software SAP2000 were investigated in the current research project by analysing actual shear wall buildings for which measured data were available. Some of the buildings listed in Table 2.1 were selected for this purpose. The buildings were then modelled for analysis, with due considerations given to the stiffness of each element and mass at each floor level. The model support conditions were considered fully fixed at the ground level. The models consisted of a series of plane frames, connected by rigid links at floor levels to impose equal horizontal displacement. The beams and columns were modelled using frame elements. The shear walls were modelled as continuous wide columns and the beam-to-wall joints were assumed to have infinite rigidity. Three degrees of freedom were allowed at each joint.

The cracking effect in members was taken into account by considering the reduced values of flexural rigidity (EI). The stiffness reduction values specified in CSA A23.3 Standard for Design of Concrete Structures [8] were used. Accordingly, 70% of the gross flexural stiffness was assigned to the columns and walls. The beam stiffnesses were reduced to 35% of their values based on gross sectional properties. These reductions are in line with those used by other researchers [7,42].

The geometric properties of buildings were taken from the available information given in the literature. When not known, the slab thickness was assumed and the beam and column dimensions were estimated from the scaled sketches provided. In some cases it was difficult to determine the dead load of a building accurately. In such cases, the mass was approximated. The mass for partitions was accounted for by considering the minimum allowance of 1.0 kPa specified in NBCC-95.

The fundamental periods of the selected structures, computed with SAP2000, and those recorded during strong motions are listed in Table 2.5. The comparison between computed and measured periods is depicted in Figure 2.9. As it can be seen, there is very good correlation between the computed and measured periods. This confirms the applicability of the analytical approach employed for computing the fundamental periods of shear wall structures.

2.5.2 Parametric investigation

The significance of flexural stiffness of shear wall buildings on the fundamental period was investigated by analysing a number of analytical models. When a structure is subjected to strong motion, it might experience inelastic deformations, and the fundamental period increases due to excessive cracking of the concrete elements. The Standard for Design of Concrete Structures (CSA-94 A23.3) recommends the use of reduced values of 70% of gross flexural stiffness for columns and walls, and 35% for beams.

Shear wall buildings of 5, 10 and 15 storeys, designed by Shooshtari [36], were chosen as representative of shear wall buildings in practice. The buildings had a symmetrical floor plan to minimise the torsional effects. The plan consisted of three bays in the transverse direction and five bays in the longitudinal direction, as shown in Figure 2.10. The shear walls were placed in the short direction. Table 2.6 provides sectional properties for columns and beams. Two-dimensional models of buildings in the short direction were established for computer analysis. The analytical buildings consisted of four interior and two exterior frames. When the frames are identical, it is a common practice to lump them for the purpose of modelling to reduce the number of elements and the time required for analysis. The interior and exterior frames were lumped to form a two-frame model where the frames were connected by rigid links to impose equal floor displacements. The rigid links were modelled using truss elements with high axial rigidity to transfer lateral loads without a moment connection between the frames. This is consistent with the rigid diaphragm assumption. Figure 2.11 shows the lumped frame model for a 10-storey building with a one-bay shear wall. The stiffness of the elements of the lumped frame is equal to the stiffness of each specific element times the number of lumped frames. For example, two identical exterior frames were modelled by one lumped exterior frame with elements that were twice as stiff as those of actual exterior frames. The support and joint conditions were modelled following the same procedure described in the previous section for the earlier set of analyses.

Six different structural wall layouts for each of the 5-, 10- and 15-storey buildings were considered for analysis. The layouts include shear walls located in the middle bay, first and third bays, and all bays. The walls were placed in two exterior frames first, and then placed in two exterior and two interior frames, as illustrated in Figure 2.12. The same wall thickness was considered for all cases. As a result, a total of 18 models with different wall layouts and heights were selected for study.

Cracking of concrete was considered as per CSA A23.3. The sectional properties were based on concrete section only and the contribution of reinforcing steel to flexural rigidity was ignored. The compressive strength of concrete was 30 MPa, and the modulus of elasticity was taken as $4500\sqrt{f'c}$ or 24,650 MPa. The dead load of the frame, excluding shear walls, was considered as 3.5 kPa for the roof and 5.0 kPa for all other floors. Mass of shear wall(s) was also considered. The live load was taken as 2.2 kPa and 2.4 kPa for the roof and floors, respectively. The storey mass, consisting of dead plus 50% of live load was lumped at floor level.

The 18 models considered earlier were analysed twice, considering 50% and 70% of the gross flexural stiffness for columns and walls; and 35% for beams. The comparison between the fundamental periods for these models is shown in Table 2.7 and Figure 2.13. The results show that a better estimation of the fundamental period of vibration is obtained when the flexural stiffness is taken as 70% for columns and walls. The flexural stiffness taken as 50% and 35% of the gross flexural stiffness for columns and beams, respectively, will account for the cracking effect induced by the earthquake at higher levels of ground motion.

Buildings with shear walls located in layouts "a", "b" and "c" have the same length of the lateral resisting system. The buildings with shear walls located in layout "c" are stiffer than in layout "a" and "b". It is evident that a better estimation of period would be obtained if the formula relies on the stiffness of the building rather than the dimension of the lateral-resisting system.

Buildings with wall layouts identified with letters "b" and "d" consist of a total of four shear walls located in two different layouts (Fig. 2.12). The mass and shear wall properties are exactly the same in both buildings. The fundamental periods for the buildings with layout "b" are 0.34, 1.15 and 2.23 seconds for the 5-, 10- and 15-storey, respectively, while the periods for layout "d" are 0.35, 1.24 and 2.50 seconds, respectively. This shows that wall layout "b" is more rigid than wall layout "d" for the 10- and 15-storey buildings and almost the same for the 5-storey building.

Similar results were obtained for the models with shear walls located in layout "c" and "f" (Fig. 2.12). The fundamental periods for the 5-, 10- and 15-storey buildings with wall layout "c" are 0.30, 1.06 and 2.28 seconds, respectively. For wall layout "f" the corresponding periods are 0.10, 0.39, and 0.85 seconds for the 5-, 10- and 15-storey buildings, respectively. As expected, the wall layout "f" is stiffer than wall layout "c". This indicates that the same number of walls located in different layout results in different values of the building stiffness and should be taken into account for an accurate computation of the fundamental period.

2.6 Improved Expressions to Estimate the Fundamental Period of Shear Wall Buildings

The empirical formulas addressed in current building codes were compared with measured data earlier in this chapter. Although some researchers suggested improved expressions for design purposes, there remains a need for further improvements in the prediction of fundamental period for shear wall buildings.

Researchers showed that the consideration of wall stiffness in empirical expressions, such as those adopted by building codes, results in better estimation of the fundamental period. As part of the current research program, improvements to existing empirical expressions are proposed and discussed in this section.

In most concrete buildings it is desirable to resist lateral loads with shear walls. Tall shear walls act as flexural cantilever elements that transfer lateral forces from the superstructure to the foundation. In order to determine the distribution of lateral loads, the lateral rigidity of shear walls should be incorporated in any analysis, as accurately as possible. The portion of total lateral force that is resisted by each wall depends on bending and shear resistances of these wall elements. From the parametric study performed in Chapter 2, it was found that the number of shear walls has crucial effect on the fundamental period of shear wall buildings.

When the wall height-to-length ratio is less than 1.0, the overall behaviour is dominated by shear, and the flexural deflection becomes negligibly small. When the aspect ratio is greater than 2.0, shear deflection can be neglected and the overall rigidity becomes proportional to the moment of inertia. For aspect ratios that vary approximately between 1.0 and 2.0, the walls behave in combined flexure and shear.

The first or fundamental mode shape of vibration is that corresponding to flexure, therefore, the fundamental period of a flexural dominant shear wall based on Rayleigh's method can be written as shown below:

$$T = \frac{2\pi}{3.516}\sqrt{\frac{m}{EI}}H^2 \tag{4.1}$$

Where, m is mass per unit length of wall, E is the modulus of elasticity and H is the height of wall. From this equation it is evident that the fundamental period of a shear wall can be expressed in terms of height-to-moment of inertia ratio. The overall effect of the moment of inertia on buildings is calculated based on the summation of the moment of inertia of individual shear walls in a given direction. This simplified method of calculating the overall stiffness of the building is validated by comparing the computed fundamental period with the field measured data. The detailed calculation of stiffness for selected buildings is presented in Appendix A-1.

Once the parameters to correlate the fundamental period were chosen for the considered shear wall structures, the next step would be to check the correlation of these parameters with measured fundamental periods of buildings. For this purpose, the selected shear wall buildings listed in Table 2.2 and the buildings for which only the wall length was known (buildings 11 to 18, in Table 2.3), were used. For the latter buildings, the wall thickness was either assumed or measured from scaled sketches. This approach did not alter the accuracy of results significantly, since the moment of inertia of a wall is not as sensitive to its thickness as it is to its length.

Several combinations of height-to-moment of inertia ratios were studied in the current investigation as follows:

$$\frac{H^{\alpha}}{I^{\beta}} \tag{4.2}$$

Where, H is the height of building in meters, I is the moment of inertia of building in m^4 and α and β are coefficients that depend on the building properties, and vary between 0.1 to 1.0. The measured periods of each of the selected building were plotted versus the height-to-moment of inertia ratios using different values for α and β. The best combination of α and β was obtained when the scattered data could be expressed with a mathematical formulation. As illustrated in Figure 2.14, the best result was obtained when α was equal to 1.0 and β was equal to 0.25. By using linear regression, the best fit of the data was obtained by the following equation:

$$T = 0.13 \frac{H}{I^{0.25}} - 0.4 \tag{4.3}$$

In this equation the parameter $H/I^{0.25}$ is dimensionless, a positive factor that makes the formula suitable for codification. This proposed formula does not account for the shear stiffness, but as it can be seen, there is a good correlation between the measured and computed fundamental periods based on the above equation. The correlation yielded a value of $R^2 = 0.9$. The R^2 value is the indicator of how well the equation developed based on regression analysis correlates with measured data. R^2=1.0 indicates that correlation obtained is perfect, and R^2=0 indicates zero correlation.

Based on these results, Equation 4.3 can be used to conservatively estimate the fundamental period of shear wall buildings.

CHAPTER 3

Fundamental Period of Reinforced Concrete Moment-Resisting Frame Buildings

3.1 General

The code-specified design lateral forces depend upon a number of factors such as weight, dimensions, and type of structures, as well as seismicity of the area in which the structure is to be built. Fundamental period of vibration is one of the crucial parameters that may have an effect on the earthquake induced lateral forces of multi-storey moment-resisting frame buildings. While it might be conservative to ignore the stiffening effect of infill panels to wind loading, disregarding this effect on the response of a structure subjected to earthquakes can be disastrous. Unless concrete or masonry walls are purposely isolated from the structural frame, the interaction of the infill walls and the frame should be considered in the structural design. Walls filling the spaces between frame members not only increase the stiffness but also change the mode of deformation of the frame under lateral loads.

This chapter report on the measured fundamental periods of moment-resisting frame buildings in the literature. Empirical expressions provided in current building codes, for fundamental period, are evaluated. Also, the effect of non-structural elements on fundamental period of moment-resisting frame buildings is presented. It was found that the information available on non-structural components of instrumented buildings is very limited. Therefore, the effect of infill walls is assessed through analytical means.

3.2 Fundamental Period Database

It is important to have field data on fundamental periods of actual buildings to assess the adequacy of the code formulas. Therefore, the fundamental period for Moment-Resisting Frame (MRF) buildings reported in the literature were collected first to establish a reliable database.

The database includes observed period values for 29 moment-resisting frame buildings in transverse and longitudinal directions, measured during the 1971 San Fernando and subsequent earthquakes. Fundamental periods of the same buildings were sometimes reported by more than one researcher. Some strong motion records obtained during the Loma Prieta and Northridge Earthquakes [28,35] were also used to obtain additional data, bringing the total number of recorded period values to 91. These data is presented in Table 3.1.

A letter and a number are used to identify each building. The letter "C", "U", "N", and "ATC" identify buildings instrumented by the California Strong Motion Instrumentation Program (CSMIP), the United States Geological Survey (USUG), the National Oceanic and Atmospheric Administration (NOAA), and buildings presented in the ATC-06 report [3]. respectively, followed by the station number. Additional information about each building, such as the number of storeys and building height, are also provided in this table.

3.3 Evaluation of the Formulas in Current Building Codes

The empirical formulas given in the 1995 edition of the National Building Code of Canada (NBCC-95) for period calculation of moment-resisting frames are as follows:

$$T = 0.1N \tag{3.1}$$

or

$$T = 0.075(h_n)^{3/4} \tag{3.2}$$

Where, N is the total number of storeys above grade and h_n is the total height of building above the base, in meters. The NBCC-95 also specifies that the fundamental period can be calculated using a rational method, such as Rayleigh's method or Eigenvalues method, but the resulting base shear shall be not less than 0.8 times the base shear calculated with the empirical formula provided in the code.

The Uniform Building Code (UBC-97) proposes the following formula for the calculation of fundamental period for moment-resisting frames:

$$T = C_t (h_n)^{3/4} \tag{3.3}$$

Where h_n is the height of the building above base in feet (or meters), and C_t is a numerical coefficient equal to 0.030 in the imperial system which corresponds to a value of 0.075 in the SI system. Since the UBC-97 formula is identical to that in NBCC-95, the evaluation of the code formulas was done only with reference to NBCC-95. The comparison between measured and NBCC-95 calculated periods is shown in Figure 3.1. The comparison includes data for both directions of buildings.

When the measured periods are plotted against number of storeys, the code equation leads to values that are much shorter than the measured periods. In some cases the measured periods are 2 or 3 times larger than the period calculated by this equation. The wide scatter of data indicates that the code equation may not be suitable for reliable estimation of the fundamental periods for moment-resisting frame buildings.

Also shown in Figure 3.1 is the comparison between measured periods and the values calculated by Equation 3.2. As it can be seen, most periods fall above the estimated code value. Few data fall under this line, indicating that the code is very conservative in estimating the fundamental period for these buildings. For buildings between 45 to 60 meters high or over 80 meters high, the code appears to underestimate measured periods in more than 50% of the cases.

3.4 Evaluation of Previously Suggested Improved Expressions

When concrete moment-resisting frame buildings are subjected to strong ground excitations, their fundamental period is lengthened because of the reduction in stiffness due to the cracking of concrete. Goel and Chopra [15] performed analyses on buildings that were strongly shaken but did not reach their yield limit. They found that the lower bound of the measured periods was about 1.2 times the code value. Therefore, it was suggested that a coefficient of 0.035 (0.086 in the SI system) be used instead of 0.030 specified in UBC-97. Bendimerad et al. [4] also suggested the same coefficient to improve the code formula. The comparison between measured and computed values based on $C_t = 0.086$ is depicted in Figure 3.2.

Goel and Chopra also proposed an equation that provided the best fit to available data. The periods for strongly and less strongly shaken buildings were also included. The proposed equation is given below:

$$T = 0.016 H^{0.9} \qquad \text{(Imperial system)} \qquad (3.4)$$

or

$$T = 0.0475 H^{0.9} \qquad \text{(SI system)}$$

Where, H is the height of the building in feet (or in meters) above the base. Based on linear regression, it was proposed that the period from a rational analysis should not exceed a value of 1.4 times the period calculated with the proposed formula.

The comparison between measured and computed values is presented in Figure 3.3. It appears that a better correlation is obtained when the code formula is used with C_t=0.086 (Fig. 3.2) rather than C_t=0.075 (Fig. 3.1). The change in this coefficient also improves the prediction for buildings that are higher than 70 meters.

The proposed formula of Goel and Chopra gives close agreement with the measured periods. The correlation seems to be almost the same as that obtained using C_t=0.086, but now a better approximation of periods is obtained for building heights ranging between 60 and 90 meters.

3.5 Analytical Assessment of Fundamental Periods for Concrete Moment-Resisting Frame Buildings

An analytical investigation was conducted to assess fundamental periods of reinforced concrete moment-resisting frame buildings. One of the objectives of the current research program was to determine the effect of non-structural elements on the fundamental period of moment-resisting frame buildings. Therefore, frame buildings were analyzed with and without non-structural infill elements. Reinforced concrete frame buildings of 5-, 10- and 15-storeys were chosen for this purpose as representative of commonly used frame structures in practice. The plan consisted of three bays in the transverse direction and five bays in the longitudinal direction, as illustrated in Fig. 3.4. A symmetrical floor plan was considered to minimize the effect of torsion. The buildings were designed previously by Shooshtari [36], based on the provisions of CSA Standard A23.3 and using SAP-90 computer program. Beam and column sections were chosen to be the same as those shown in Table 2.6. Concrete compressive strength of f'_c=30 MPa was used in design. Clay brick masonry infill walls were assumed as non-structural elements. Two different grades of masonry were considered: NW with a minimum compressive strength of 8.6 MPa; and MW with a minimum compressive strength of 15.2 MPa. The wall thickness was either equal to 100 mm and 200 mm. The infill walls were located in one, two or three bays of the exterior frames (Fig. 3.4).

3.5.1 Analytical Models

Two-dimensional analytical models were developed for analysis using SAP200 Computer Software. The frames with identical properties were lumped together, and the lumped frames were connected by rigid links to impose equal horizontal displacement at each floor level, without any moment connection between the frames (Fig. 3.5). It was assumed that the floor system would act as a rigid diaphragm. The supports were fully fixed against rotation at the base. Three degrees of freedom were contemplated at each node. The beam–column connections were assumed to be infinitely rigid.

Concrete cracking in members was taken into account by reducing the gross stiffness values. Accordingly, 70% of the gross flexural stiffness for columns, and 35% for beams were used. This is consistent with the recommendations of CSA Standard A23.3. The mass was lumped at each storey level, including the mass of infill walls, when present. The wall-area-to-floor-area ratio, R, was calculated for each case as a parameter reflecting the stiffening effect of walls. This ratio is defined as the plan area of the infill (thickness x length) divided by the floor plan area.

3.5.2 Modeling of Infill Walls

At moderate levels of lateral load, there is no contact between the infill wall and the frame due to the deformation of frame, except at the corners where the compression forces are transmitted through diagonal struts. Based on this concept, a simple method was developed by Stafford-Smith [4] to determine the stiffness of an infill wall. In this method, the geometric properties of the equivalent strut depend upon the length of the contact surface between the wall and the columns, α_L, and between the wall and the beams, α_h . These properties are illustrated in Figures 3.6 and 3.7 and are calculated as follows:

$$\alpha L = \frac{\pi}{2} \sqrt[4]{\frac{4\ E_f \cdot I_c \cdot h}{E_m \cdot t \cdot \sin 2\theta}} \qquad (3.5)$$

and

$$\alpha h = \pi \sqrt[4]{\frac{4\ E_f \cdot I_b \cdot L}{E_m\ \cdot t\ \cdot \sin 2\theta}} \tag{3.6}$$

Where, E_m and E_f are the elastic moduli of the masonry and frame materials, respectively; h, t, L are the height, thickness and length of infill wall, respectively; I_c and I_b are the moments of inertia of columns and beams, respectively; and θ is the angle of strut with respect to the horizontal axis, defined as $\tan^{-1}$ (h/L). The modulus of elasticity of masonry, E_m, is calculated as:

$$E_m = K\, f_m \tag{3.7}$$

Where, f_m is the compressive strength of masonry in MPa, and K is equal to 750 for concrete block and 500 for clay brick. The modulus of elasticity of concrete frame is taken as $4500\sqrt{f'_c}$. The equivalent or effective width of strut is determined as follows, assuming that the strut is subjected to uniform stress:

$$w = \frac{1}{2}\ \sqrt{\alpha_h^2 + \alpha_L^2} \tag{3.8}$$

Alternatively, the New Zealand Building Code specifies the effective width equal to one quarter of the length of the strut, L_d. Holmes [18] suggests an effective width of one third of L_d. The properties of equivalent struts are summarized in Table 3.2.

The effective width of the compression strut calculated according to Holmes and the New Zealand Building Code are compared to those calculated by Equation 3.8. The fundamental period of the 10-storey building was then computed using the different effective widths. The difference in fundamental period for the cases considered is small, as shown in Table 3.3. The calculated fundamental periods varied only about 2% when different effective widths were considered. The equivalent strut width throughout this research was calculated based on Equation 3.8.

3.5.3 Effect of Non-Structural Elements on Fundamental Period

The effect of non-structural elements on the fundamental period of buildings is often ignored in current building code formulas. Their effects were studied analytically, using the structures described earlier. Specifically, the effect of wall thickness, the number of bays containing infill walls (or length of the infill) and the strength of infill material have been examined.

The increase in stiffness is expressed in terms of the wall-area-to-floor-area ratio, R. As this ratio (R) increased, the fundamental period of the frame decreased. Table 3.4 summarizes fundamental periods obtained with the different infill wall thickness' and masonry strengths for the 5-, 10- and 15-storey buildings. Also shown in this table are the corresponding R-values for each case.

Figure 3.8 shows the effect of infill walls in a 5-storey building. The building with 8.6 MPa masonry strength is denoted by a dotted line, while that of 15.2 MPa masonry is denoted by a solid line. Also shown in this figure is the calculated period based on the NBCC-95 formula. Similar comparisons are presented in Figures 3.9 and 3.10 for 10-storey and 15-storey buildings, respectively. The fundamental periods of 5-, 10- and 15-storey buildings with wall thickness of 100 and 200 mm, are compared in Figures 3.11 and 3.12, respectively. The effect of each parameter is discussed as follows based on these analytical results:

a. Effect of infill wall thickness: The fundamental period was reduced significantly by adding a single bay infill wall panel in all buildings, even if it was only 100 mm thick. For the 5-storey building, the reduction in period was 47%, as compared to that of bare frame building. The reduction was 40% in the 10-storey building and 34% in the 15-storey building. When the wall thickness was increased by a factor of 2 (t=200 mm), the period for the 5-storey building was reduced by additional 6% down to 53%. Similarly, small shortening was observed in the periods of 10- and 15-storey buildings when the wall thickness was increased to 200 mm, resulting in 44% and 36% reductions in period, respectively, as compared to those of bare frame buildings. These comparisons indicate that a change in wall thickness does not translate in proportional changes in the fundamental period.

b. Effect of masonry strength: The effect of masonry strength on structural period was investigated by considering two different types of masonry. In the case of 100 mm thick infill walls, when 8.6 MPa masonry units were used in a single bay of the 5-storey building, the computed period was 0.92 seconds. When 15.2 MPa units were used instead, the computed period was 0.83 seconds. This indicates only a 10% reduction in the fundamental period. The 5-storey building infilled in two and three bays with 15.2 MPa masonry units resulted in period reductions of 13% and 14%, respectively, as compared to the corresponding building with 8.6 MPa units. For the 10-storey building the fundamental period reductions were 5%, 8% and 10% when the frames were infilled in one, two and three bays, respectively. The corresponding reductions for the 15-storey building were 3%, 3% and 7%, respectively. Similar results were obtained with 200 mm thick infill walls of different strengths. The effect of masonry strength is depicted in Figures 3.8, 3.9 and 3.10. The masonry strength appears to have only a small impact on the fundamental period and therefore, it can be neglected for period computation.

c. Effect of number of bays infilled with masonry walls: The major impact on period occurred when a single panel was used, corresponding to R=0.0021 and R=0.0042 for 100mm and 200mm thick walls. This is shown in Figures 3.8, 3.9 and 3.10. As this ratio increased the building period decreased by about 60% to 70% as compared to bare frame structure. For the 5-storey building, the period of vibration was reduced by 44% to 46%, depending on the thickness or strength of the infill. For the 10-storey building the reduction was about 40% to 46%, while for the 15-storey buildings was from 34% to 38%. When two bays were filled with masonry walls, a reduction of 61% to 70% occurred in the 5-storey building. However, this reduction is only about 15% more than where only one bay was infilled. The reduction in the 10- and 15-storey buildings was about 16%. Similarly, when three bays were infilled, the fundamental periods were reduced by 9% and 10% for the 10- and 15-storey buildings, respectively.

d. Comparison with NBCC-95: There is a very good agreement between the NBCC-95 predictions and the analytical fundamental periods for buildings of 40 to 60 m. height range when exterior frames are fully infilled with masonry walls (R=0.0062 and R=0.0124 in the current research). The computed values vary only 3% to 10% compared to the code periods. As expected, the observed trend indicates that when the computed periods approach the code's estimation, further stiffening of the structure would not affect the fundamental period considerably (Figs. 3.8 to 3.10). However, verification of the fundamental period of analytical models with existing building models is still required.

3.6 Improved Expressions to Estimate the Fundamental Period of Moment-Resisting Frame Buildings

Masonry walls are frequently used in exterior enclosure and interior partitioning of residential, commercial and industrial buildings. Recent earthquakes have demonstrated that the presence of masonry infills affects the seismic performance of reinforced concrete buildings. Experimental investigations, as well as actual field observations clearly indicate that masonry infills provide diagonal compression struts and contribute to the overall strength and stiffness of frame structures, reducing the period of vibration.

From the analytical study presented previously in this chapter, it was shown that infill panels with a wall-to-floor-area ratio of R=0.0021 were sufficient to reduce the fundamental period by 40%. Therefore, in this study, an empirical expression has been developed for the computation of the fundamental period, with due considerations given to the effects of non-structural elements.

As described previously, 39 analytical models were considered to study the effect of the infilled masonry walls on the fundamental period of the frames. First the fundamental periods of the bare frames were calculated and then the periods of the infilled frames were obtained. The computed periods for the 5-, 10- and 15- storey buildings are shown in Table 3.4. The comparison of the results of each building is depicted in Figs. 3.11 and 3.12.

From the analytical study performed in the current research, it was shown that the thickness of the infill wall and the masonry strength used had little effect on the fundamental period of the moment-resisting frame. Therefore, by ignoring these parameters, the fundamental period of an infilled frame could be expressed as a function of the height of the building (H) and the wall-to-floor-area ratio only, T= f (H, R).

Using the linear regression method, an equation was found to express the fundamental period of the infilled frame. The suggested equation is given as:

$$T = 0.008\mathrm{H}^{1.5}(1 - 40\mathrm{R}) \tag{4.4}$$

where $0.002 < \mathrm{R} < 0.012$

In this equation R is the wall-to-floor-area-ratio calculated as explained previously and H is the height of the building, in meters. The comparison between the suggested equation and the analytical models are shown in Figure 3.13. As it can be seen in this graph, there is a very good correlation between the computed values the suggested values indicating that this equation can be used to estimate the fundamental period of the infilled frames.

Although it is essential to validate the suggested equation by using measured data, this could not be done since the layout and numbers of infills in the frames for instrumented buildings were not available. Therefore, this portion of the current investigation was based entirely on the analytical results.

CHAPTER 4

Conclusions

4.1 Reinforced Concrete Shear Wall Buildings

Based on the investigation carried out in this research, the following conclusions can be drawn:

1. The fundamental period of shear wall buildings can be expressed for design purposes in terms of the $H/I^{0.25}$ ratio. The proposed equation: $T = 0.13\ H/I^{0.25} - 0.4$ provides a better approximation of the period than those suggested by current building codes. In this formulation, the moment of inertia of the building is calculated as the summation of the moments of inertia of all individual walls. The $H/I^{0.25}$ ratio is dimensionless.

2. Based on the comparison of field measured periods of 34 shear wall buildings, with those computed on the basis of UBC-97 and NBCC-95, it can be concluded that a better estimation is obtained when coefficient C_t that relates the fundamental period to building height in metres is 0.0488. This coefficient was found to produce more accurate results than those obtained using the detailed alternative approach presented in UBC-97, where $C_t = 0.1/\sqrt{A_c}$. In general, the values obtained by UBC-97

expressions showed better correlation with measured data than those computed based on NBCC-95.

3. The equation proposed by Goel and Chopra (Eq. 2.5) for shear wall structures, provided improved results as compared to any of the code-suggested expressions.

4. The expression in NBCC-95, which is a function of the length of lateral load resisting system (D_s), is difficult to use since D_s is not easily identifiable in most practical applications. In some cases this approach resulted in non-conservative estimation of fundamental period. Several alternative approaches were considered to identify the appropriate length for D_s in the current research project. The best estimation of period values was obtained when the length of lateral load resisting system was taken as the summation of individual shear wall lengths of all frames divided by the number of frames containing these shear walls.

5. Ignoring the coupling effect of shear walls when calculating the fundamental period using the expression given in NBCC-95, resulted in the underestimation of the actual stiffness of the building. Considering the shear walls to be coupled when the distance between them is equal or less than one half the length of the shorter wall, and using the length of the entire coupled wall system in establishing D_s, improved the fundamental period calculation.

6. Reinforced concrete shear walls can be modelled as continuous wide columns and analysed for period determination, provided that the wall stiffness is reduced to 70% of the gross value. The results obtained from such analysis agree very well with those obtained from field data.

4.2 Reinforced Concrete Moment-Resisting Frame Buildings

The following conclusions can be made based on the research conducted on moment-resisting frame buildings:

1. The empirical expression given in UBC-97 and NBCC-95, in the form of $T= C_t H^{3/4}$ with $C_t= 0.075$ (when H is in meters), produces better estimation of the fundamental period than the alternative simple formula T= 0.1N, where N is the number of storeys.

2. The computation of fundamental period with UBC-97 and NBCC-95 equation was improved when the coefficient $C_t= 0.086$ (0.035 in imperial system) was used instead of $C_t= 0.075$ (0.03 in imperial). This also improved the estimation of periods for buildings of more than 70 m, which were underestimated with $C_t= 0.075$. The improved coefficient produced results that were similar to those obtained by the expression proposed by Goel and Chopra (Eq. 3.4) up to 60 meter height. However, the period values produced by Goel and Chopra's expression produced better results in the 60 m to 90 meter height range.

3. Non-structural elements, in the form of masonry infill walls, have a significant effect on fundamental periods of moment-resisting frames. For buildings in the 20 to 60 meter height range, the fundamental period was shortened by 35% to 75% because of the stiffening effect of infill walls. In this research, the effects of infill walls were incorporated in terms of wall area-to-floor area ratio (R). It was found that the effect of infill walls could be introduced through Eq. 3.9.

4. The stiffness of infilled frame is affected by wall thickness and masonry strength. As the wall area-to-floor area ratio (R) increases, the stiffening effect of infill walls becomes more significant. The increase in stiffness due to wall thickness has little effect on fundamental period. This effect was found to be slightly more significant on low-rise buildings. The impact of masonry strength on building stiffness and hence period, was found to be negligible. The masonry strength plays slightly more important role on periods of low-rise buildings.

Building ID		No. of stories	Height (m)	Width (m)	Length (m)	Period T (s)		Earthquake
						Long.	Transv.	
Reinforced Concrete Shear Walls								
Belmont	C58262	2	8.50	N/A	N/A	0.13	0.20	Loma Prieta
Burbank	C24385	10	26.6	22.7	65.0	0.60	0.56	Northridge
Burbank	C24385	10	26.6	22.7	65.0	0.57	0.51	Whittier
Burbank *	C24385	10	26.6	22.7	65.0	0.62	0.57	Northridge
Burbank	C24385	10	26.6	22.7	65.0	0.69	0.63	Northridge
El Centro, ICSB	N/A	6	24.8	25.8	41.4	MRF	0.50	Imperial Valley
El Centro, ICSB	N/A	6	24.8	25.8	41.4	MRF	0.44	Imperial Valley
Goleta	C25213	3	10.0	N/A	N/A	0.30	0.35	Santa Barbara
Hayward	C58488	4	15.0	N/A	N/A	0.15	0.22	Loma Prieta
Long Beach	C14311	5	21.5	24.5	62.0	0.17	0.34	Whittier
Los Angeles	ATC-3	12	48.1	18.2	49.0	1.15	MRF	San Fernando
Los Angeles *	C24468	8	38.5	19.0	46.6	1.54	1.62	Northridge
Los Angeles *	C24601	17	45.3	24.2	68.7	1.18	1.05	Northridge
Los Angeles	C24601	17	45.3	24.2	68.7	1.00	1.00	Sierra Madre
Los Angeles	C24601	17	45.3	24.2	68.7	0.80	1.00	Northidge
Los Angeles	N253-5	12	49.0	23.0	47.0	1.07	1.13	San Fernando
Los Angeles	N253-5	12	49.0	23.0	47.0	1.19	1.14	San Fernando
Los Angeles *	C24665	6	18.5	78.0	92.5	0.40	0.50	Northridge
Newport Beach †	C13589	11	44.5	23.3	46.6	0.82	0.87	Northridge
Oakland	C58483	24	66.3	23.6	64.0	2.32	3.23	Loma Prieta
Palm Desert *	C12284	4	15.8	18.0	54.5	0.50	0.60	Palm Spring
Pasadena	N264-5	10	43.0	21.0	22.7	0.71	0.52	Lytle Creek
Pasadena	N264-5	10	43.0	21.0	22.7	0.98	0.62	San Fernando
Piedmont *	C58334	3	11.0	21.7	34.4	0.18	0.18	Loma Prieta
Pleasant Hill *	C58348	3	12.3	23.3	39.7	0.38	0.46	Loma Prieta
Pleasant Hill	C58348	3	12.3	23.3	39.7	0.38	0.56	Loma Prieta
San Bernardino †	C23285	5	20.3	41.3	62.0	0.49	0.64	Northridge

N/A refers to data not available; MRF=Moment Resisting Frame

* Indicates the selected shear wall buildings

† Indicates selected buildings for which the shear wall thickness was assumed

Table 2.1 Measured fundamental periods of vibration of buildings (Continues…)

Building ID		No. of stories	Height (m)	Width (m)	Length (m)	Period T (s)		Earthquake
						Long.	Transv.	
San Bernardino	C23287	6	15.5	17.6	52.0	0.53	0.25	Northridge
San Bruno *	C58394	9	31.5	25.4	58.0	1.20	1.30	Loma Prieta
San Bruno	C58394	9	31.5	25.4	58.0	1.00	1.45	Loma Prieta
San Bruno	C58394	9	31.5	25.4	58.0	0.86	1.07	Loma Prieta
San Francisco †	C58479	6	24.0	20.3	55.0	0.50	0.22	Loma Prieta
San Jose *	C57355	10	37.5	24.8	57.5	MRF	0.75	Loma Prieta
San Jose	C57355	10	37.5	24.8	57.5	MRF	0.70	Loma Prieta
San Jose	C57355	10	37.5	24.8	57.5	MRF	0.61	Morgan Hill
San Jose	C57355	10	37.5	24.8	57.5	MRF	0.61	Mount Lewis
San Jose	C57356	10	29.0	19.4	63.6	0.73	0.43	Loma Prieta
San Jose *	C57356	10	29.0	19.4	63.6	0.70	0.42	Loma Prieta
San Jose	C57356	10	29.0	19.4	63.6	0.65	0.43	Morgan Hill
San Jose	C57356	10	29.0	19.4	63.6	0.63	0.41	Mount Lewis
Saratoga †	C58235	1	10.0	37.0	46.6	0.31	0.18	Loma Prieta
Watsonville †	C47459	4	20.0	21.5	22.7	0.24	0.35	Loma Prieta
Whittier	N/A	10	27.3	19.4	55.5	MRF	0.60	Whittier
Whittier †	N/A	10	27.3	19.4	55.5	MRF	0.60	Whittier Narrows
Whittier †	N/A	10	27.3	19.4	55.5	MRF	0.60	Whittier Narrows
Masonry Shear Walls								
Concord	C58492	8	22.7	N/A	N/A	0.74	0.38	Loma Prieta
Lancaster †	C24517	3	12.6	21.0	39.0	0.21	0.20	Whittier
Palo Alto	C58264	2	7.2	N/A	N/A	0.27	0.34	Loma Prieta
Whittier †	C14606	8	23.0	15.7	21.2	0.68	0.55	Northridge
MRF and Reinforced Concrete Shear Walls								
Richmond	C58503	3	11.3	N/A	N/A	0.27	0.29	N/A
Santa Rosa	C68489	14	37.5	24.2	26.0	0.50	0.59	N/A
Santa Rosa	C68489	14	37.5	24.2	26.0	1.01	1.19	N/A
Walnut creek	C58364	10	39.0	31.5	44.8	0.51	0.72	N/A

N/A refers to data not available; MRF=Moment Resisting Frame

* Indicates the selected shear wall buildings

† Indicates selected buildings for which the shear wall thickness was assumed

Table 2.1 Measured fundamental periods of vibration of buildings (...Continued)

H (m)	Transverse D_s (m)	Transverse $T_{NBCC-95}$ (s)	Transverse $T_{measured}$ (s)	Longitudinal D_s (m)	Longitudinal $T_{NBCC-95}$ (s)	Longitudinal $T_{measured}$ (s)
Los Angeles, Administration building C24468						
38.50	1.91 [1]	2.51	1.62	3.98 [1]	1.74	1.54
	5.78 [2]	1.44		3.98 [1]	1.74	
	2.41 [3]	2.23		6.63 [3]	1.35	
	11.57 [4]	1.02		6.63 [3]	1.35	
	16.40 [5]	0.86		13.90 [5]	0.93	
Los Angeles, Parking building C24655						
18.47	9.80 [1]	0.53	0.50	21.80 [1]	0.36	0.40
	29.40 [3]	0.31		21.80 [3]	0.36	
	29.40 [5]	0.31		21.80 [5]	0.36	
Piedmont, School building C58334						
10.90	4.90 [1]	0.44	0.18	4.90 [1]	0.44	0.18
	9.80 [3]	0.31		9.80 [3]	0.31	
	9.80 [5]	0.31		9.80 [5]	0.31	
Pleasant Hill, Commercial building C58348						
12.30	5.80 [1]	0.46	0.46	6.00 [1]	0.45	0.38
	5.80 [3]	0.46		12.00 [3]	0.32	
	5.80 [5]	0.46		12.00 [5]	0.35	

Description of D_s:
1. Total length of shear walls in plan over the total number of shear walls
2. Total length of shear walls in plan over the total number of shear walls, excluding the shear walls between gridlines F-G and 1-23 (refer to appendix A-1 for plan details)
3. Summation of the length of shear walls in all frames over number of frames containing shear walls
4. Summation of the length of shear walls in all frames over number of frames containing shear walls, exlcuding the shear walls between gridlines F-G and 1-23 (refer to appendix A-1 for plan details)
5. Largest sum of the length of shear walls in any frame

Table 2.2 Comparison of measured periods and the NBCC-95's considering different approaches to estimate D_s in buildings with isolated shear walls

H (m)	Transverse			Longitudinal		
	D_s (m)	$T_{NBCC-95}$ (s)	$T_{measured}$ (s)	D_s (m)	$T_{NBCC-95}$ (s)	$T_{measured}$ (s)
Burbank, Residential building C24385						
26.60	7.81 [1]	0.86	0.57	8.60 [1]	0.82	0.62
	14.37 [2]	0.63		12.06 [2]	0.69	
	14.06 [3]	0.64		30.15 [3]	0.44	
	15.81 [4]	0.60		30.15 [4]	0.44	
	19.50 [5]	0.54		38.00 [5]	0.39	
Los Angeles, Residential building C24601						
45.30	9.02 [1]	1.36	1.05	9.61 [1]	1.32	1.18
	13.37 [2]	1.12		19.19 [2]	0.93	
	19.45 [3]	0.92		23.06 [3]	0.85	
	15.81 [4]	1.03		26.86 [4]	0.79	
	22.25 [5]	0.86		47.60 [5]	0.59	
San Jose, Residential building C57356						
29.00	21.90 [1]	0.56	0.42	7.30 [1]	0.97	0.70
	25.50 [2]	0.52		7.30 [2]	0.97	
	14.09 [3]	0.70		21.90 [3]	0.56	
	15.30 [4]	0.67		25.50 [4]	0.52	
	17.46 [5]	0.62		21.90 [5]	0.56	

Description of D_s:

1. Total length of shear walls in plan over the total number of shear walls
2. Total length of shear walls (coupled shear wall definition taken into account where applicable) in plan over the total number of shear walls
3. Summation of the length of shear walls in all frames over number of frames containing shear walls
4. Summation of the length of shear walls (coupled shear wall definition taken into account where applicable) in all frames over the number of frames containing shear walls
5. Largest sum of the length of shear walls in any frame (coupled shear wall definition taken into account where applicable)

Table 2.3 Comparison of measured periods and the NBCC-95's considering different approaches to estimate D_s in buildings with coupled shear walls

H (m)	Transverse			Longitudinal		
	D_s (m)	$T_{NBCC-95}$ (s)	$T_{measured}$ (s)	D_s (m)	$T_{NBCC-95}$ (s)	$T_{measured}$ (s)
Palm Desert, Medical building C12284						
15.20	4.80 [1]	0.62	0.60	3.87 [1]	0.70	0.50
	4.80 [2]	0.62		9.03 [2]	0.46	
	6.70 [3]	0.53		14.5 [3]	0.36	
San Bruno, Office building C58394						
31.50	5.75 [1]	1.18	1.30	4.60 [1]	1.32	1.20
	5.75 [2]	1.18		11.50 [2]	0.84	
	5.75 [3]	1.18		14.50 [3]	0.74	

Description of D_s:
1. Total length of shear walls in plan over the total number of shear walls
2. Summation of the length of shear walls in all frames over number of frames containing shear walls
3. Largest sum of the length of shear walls in any frame

Table 2.4 Comparison of measured periods and the NBCC-95's considering different approaches to estimate D_s in buildings with shear walls in a central core

Building ID	Height (m)	Measured Period (s)		Computed Period (s)	
		Transverse	Longitudinal	Transverse	Longitudinal
Burbank C24385	26.60	0.57	0.62	0.71	0.52
Los Angeles C24468	38.50	1.62	1.54	1.06	1.63
Los Angeles C24601	45.30	1.05	1.18	1.21	1.11
Palm Desert C12284	15.20	0.60	0.50	0.37	0.42
Piedmont C58334	10.90	0.18	0.18	0.25	0.25
Pleasant Hill C58348	12.30	0.46	0.38	0.38	0.26
San Bruno C58394	31.50	1.30	1.20	1.36	1.07
San Bruno C58394	31.50	1.45	1.00	1.36	1.07
San Jose C57356	37.50	0.45	MRF	0.68	MRF
San Jose C57355	29.00	0.42	0.70	0.64	1.17
Los Angeles C24665	18.50	0.50	0.26	0.63	0.42

MRF= Moment Resisting Frame

Table 2.5 Fundamental periods for selected shear wall buildings computed with SAP2000

Localtion	Level	Beam size (mm x mm)	Column size (mm x mm)	
			Interior	Exterior
Interior Frame	All floors except roof	300 x 450	500x 500	450 x 450
	Roof	300 x 400	500x 500	450 x 450
Exterior Frame	All floors except roof	300 x 450	400 x 400	350 x 350
	Roof	300 x 400	400 x 400	350 x 350

Table 2.6 Typical beam and column cross-sectional dimensions used in the analytical models

Shear wall layout †	**5- storey building**		**10- storey building**		**15- storey building**	
	$0.7I_g$ [1]	**$0.5I_g$ [2]**	**$0.7I_g$ [1]**	**$0.5I_g$ [2]**	**$0.7I_g$ [1]**	**$0.5I_g$ [2]**
a	0.46	0.54	1.55	1.75	2.99	3.30
b	0.34	0.40	1.15	1.32	2.23	2.46
c	0.30	0.35	1.06	1.23	2.28	2.60
d	0.35	0.41	1.24	1.42	2.50	2.80
e	0.22	0.26	0.72	0.81	1.72	1.97
f	0.10	0.10	0.39	0.46	0.85	1.00

† Refer to Figure 2.12 for details

1. Fundamental periods computed using the reduced flexural stiffness as 70% and 35% of the gross flexural stiffness for columns and beams, respectively
2. Fundamental periods computed using the reduced flexural stiffness as 50% and 35% of the gross flexural stiffness for columns and beams, respectively

Table 2.7 Effect of the reduced flexural stiffness on the fundamental period of shear wall buildings

Building ID		No. of Stories	Height (m)	Period T (s)		Earthquake
				Long.	Transv.	
Emeryville	N/A	30	90.0	2.80	2.80	Loma Prieta
Los Angeles	N/A	9	36.0	1.40	1.30	San Fernando
Los Angeles	N/A	14	48.0	1.80	1.60	San Fernando
Los Angeles	N/A	13	50.0	1.90	2.40	San Fernando
Los Angeles	ATC-12	10	42.0	1.40	1.60	San Fernando
Los Angeles	ATC-14	7	18.0	0.90	1.20	San Fernando
Los Angeles	ATC-2	7	21.0	1.00	1.00	San Fernando
Los Angeles	ATC-3	12	48.0	SW	1.33	San Fernando
Los Angeles	ATC-5	19	60.0	2.15	2.22	San Fernando
Los Angeles	ATC-6	11	37.5	1.43	1.60	San Fernando
Los Angeles	ATC-7	22	62.0	1.90	2.20	San Fernando
Los Angeles	ATC-9	16	46.0	1.10	1.80	San Fernando
Los Angeles	ATC-8	N/A	65.0	1.00	1.62	San Fernando
Los Angeles	ATC-10	N/A	50.0	1.10	1.30	San Fernando
Los Angeles	ATC-11	N/A	36.0	0.70	1.22	San Fernando
Los Angeles	ATC-13	N/A	42.0	0.60	0.92	San Fernando
Los Angeles	C24236	14	45.0	N/A	2.28	Northridge
Los Angeles	C24236	14	45.0	1.69	2.27	Northridge
Los Angeles	C24463	5	36.0	1.61	1.46	Northridge
Los Angeles	C24463	5	36.0	1.40	1.30	Whittier
Los Angeles	C24463	5	36.0	1.46	1.37	Northridge
Los Angeles	C24569	15	83.0	3.11	3.19	Northridge
Los Angeles	C24579	9	43.0	1.39	1.28	Northridge
Los Angeles	C24579	9	43.0	1.71	1.28	Northridge
Los Angeles	N220-2	20	60.0	2.27	2.09	San Fernando
Los Angeles	N220-2	20	60.0	2.27	2.13	San Fernando
Los Angeles	N220-2	20	60.0	2.24	1.98	San Fernando
Los Angeles	N446-8	22	62.0	1.94	2.14	San Fernando
Los Angeles	N446-8	22	62.0	1.84	2.17	San Fernando

N/A= not available; SW= Shear Walls

Table 3.1 Fundamental periods of vibration for reinforced concrete moment-resisting frame buildings (Continues...)

Building ID		No. of Stories	Height (m)	Period T (s)		Earthquake
				Long.	Transv.	
North Hollywood	C24464	20	51.0	2.60	2.62	Northridge
North Hollywood	C24464	20	51.0	2.15	2.21	Whittier
North Hollywood	C24464	20	51.0	2.20	2.60	Northridge
Pomona	C23511	2	9.0	0.28	0.30	Upland
Pomona	C23511	2	9.0	0.27	0.29	Whittier
Pomona	C23511	2	9.0	0.70	0.80	Whittier
San Bruno	C58490	6	24.0	0.85	1.10	Loma Prieta
San Bruno	C58490	6	24.0	0.85	1.02	Loma Prieta
San Jose	C57355	10	37.0	1.01	SW	Loma Prieta
San Jose	C57355	10	37.0	0.91	SW	Morgan Hill
San Jose	N/A	5	20.0	0.83	0.83	Morgan Hill
Sherman Oaks	ATC-4	13	38.0	1.20	1.40	San Fernando
Sherman Oaks	C24322	13	56.0	1.90	2.30	Whittier
Sherman Oaks	C24322	13	56.0	N/A	2.44	Whittier
Sherman Oaks	C24322	13	56.0	2.60	2.90	Northridge
Van Nuys	ATC-1	7	20.0	0.79	0.88	San Fernando
Van Nuys	C24386	7	20.0	1.40	1.20	Whittier
Van Nuys	C24386	7	20.0	1.40	1.30	Northridge
Van Nuys	C24386	7	20.0	2.08	1.94	San Fernando

N/A= not available; SW= Shear Walls

Table 3.1 Fundamental periods of vibration for reinforced concrete moment-resisting frame buildings (...Continued)

Masonry strength (MPa)	Infill wall dimensions thickness (mm)	height (m)	length (m)	αh (m)	αL (m)	Equivalent diagonal strut width (m)
8.6	100	3.55	5.60	1.85	4.21	2.30
	200			1.55	3.54	1.93
15.2	100	3.55	5.6	1.60	3.67	2.00
	200			1.35	3.07	1.68

Table 3.2 Equivalent diagonal strut properties calculated using Eq. 3.8

Masonry Strength (MPa)	Infill wall thickness (mm)	Stafford-Smith		Holmes		New Zealand Code	
		strut width (m)	T (s)	strut width (m)	T (s)	strut width (m)	T (s)
8.6	100	2.30	2.21	2.21	2.22	1.66	2.32
	200	1.93	2.10		2.08		2.14
15.2	100	2.00	2.09	2.21	2.07	1.66	2.14
	200	1.68	2.02		1.98		2.03

Table 3.3 Fundamental periods for the 10-storey moment resisting frame building using different infill equivalent strut widths

No. of Storeys	Infill wall location	Infill wall thickness (mm)	$R=A_w/A_f$	Fundamental period infilled frames (s)	
				f'_m=8.6 MPa	f'_m=15.2 MPa
5 *H=20 m.*		100	0	1.74	1.74
	1 bay		0.0021	0.92	0.83
	2 bays		0.0042	0.68	0.59
	3 bays		0.0062	0.56	0.48
		200	0	1.73	1.73
	1 bay		0.0042	0.82	0.75
	2 bays		0.0083	0.59	0.52
	3 bays		0.0124	0.48	0.41
10 *H=40 m.*		100	0	3.7	3.7
	1 bay		0.0021	2.21	2.09
	2 bays		0.0042	1.62	1.49
	3 bays		0.0062	1.29	1.16
		200	0	3.7	3.7
	1 bay		0.0042	2.09	2.01
	2 bays		0.0083	1.5	1.41
	3 bays		0.0124	1.17	1.08
15 *H=60 m.*		100	0	5.7	5.7
	1 bay		0.0021	3.75	3.63
	2 bays		0.0042	2.86	2.73
	3 bays		0.0062	2.29	2.14
		200	0	5.7	5.7
	1 bay		0.0042	3.64	3.56
	2 bays		0.0083	2.76	2.66
	3 bays		0.0124	2.17	2.07

Table 3.4 Fundamental periods of the moment resisting frame models with infill masonry walls

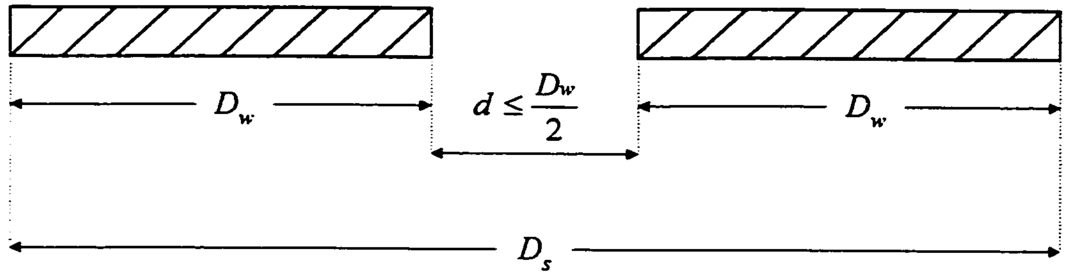
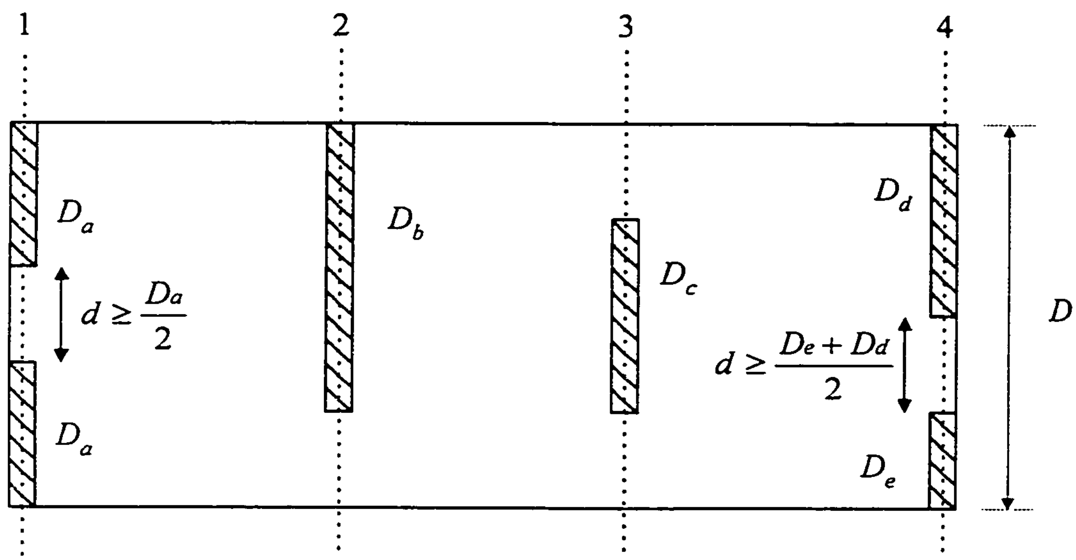


Fig. 2.1a Definition of dimension D_s for coupled shear walls in this research



Grid line 1: $D_1 = D_a + D_a$
Grid line 2: $D_2 = D_b$
Grid line 3: $D_3 = D_c$
Grid line 4: $D_4 = D_d + D_e$
Number of grid lines: 4

Average of length of shear walls per grid line:
$D_s = (D_1+D_2+D_3+D_4) / 4$

Fig. 2.1b Definition of dimension D_s for isolated shear walls

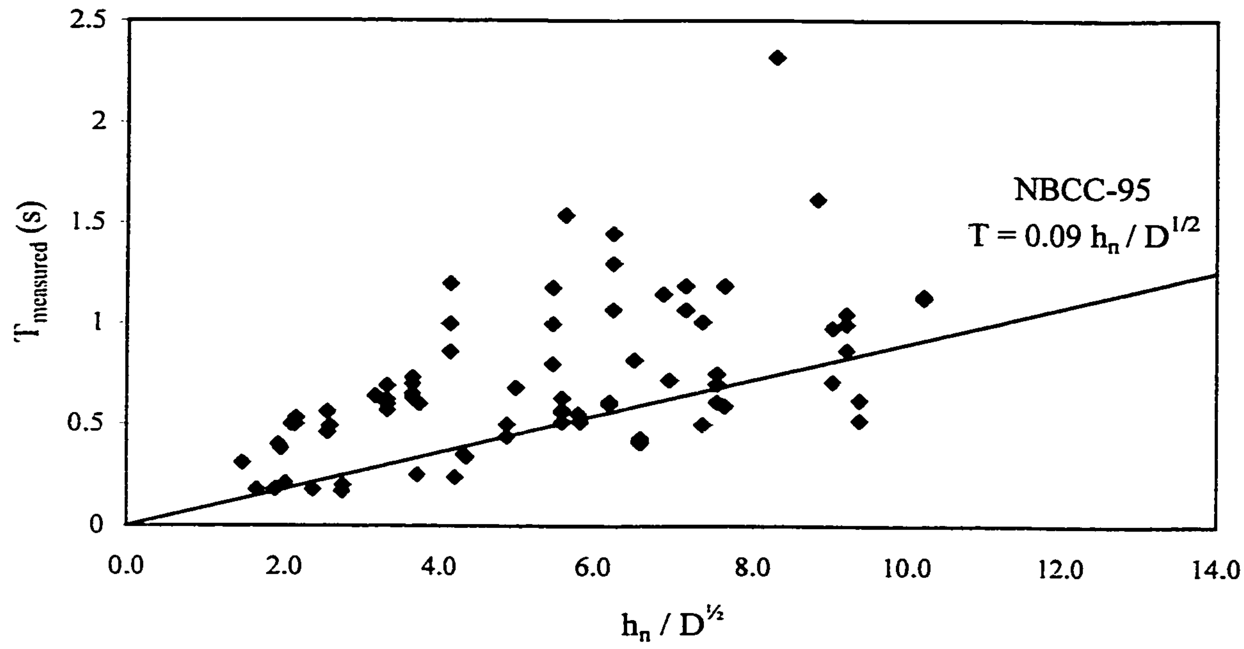


Fig. 2.2 Comparison between measured periods and the NBCC-95 estimated periods of existing shear wall buildings using the dimension D instead of D_s

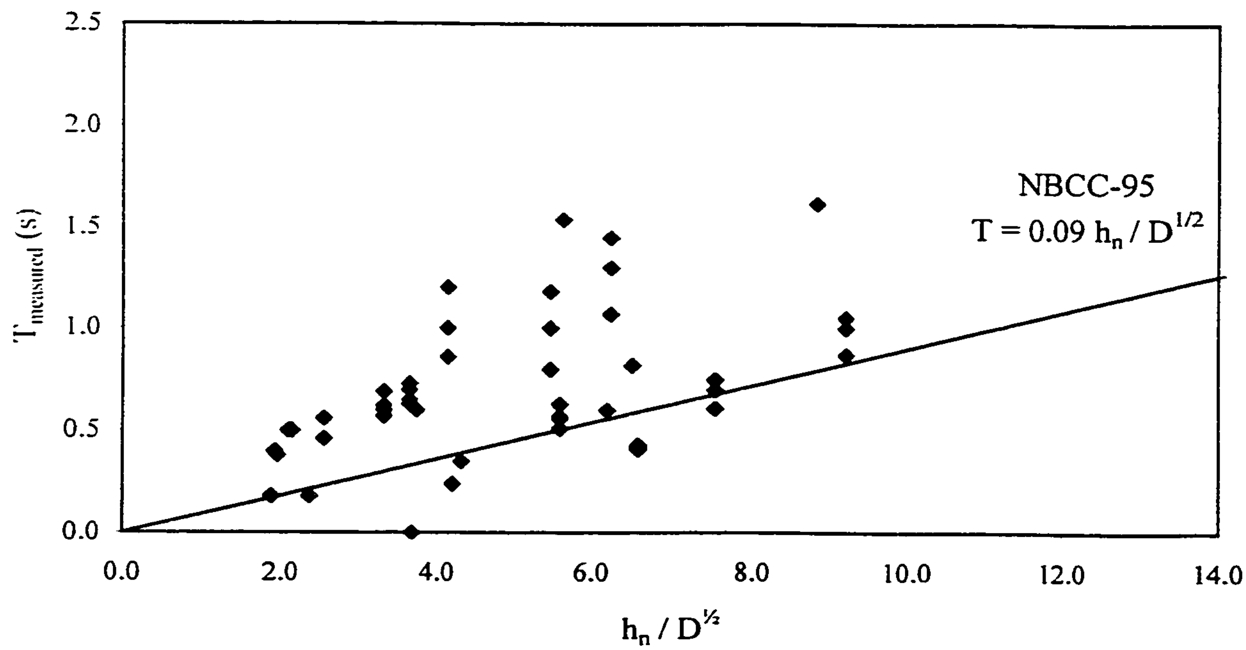


Fig. 2.3 Comparison between measured periods of selected shear wall buildings and the NBCC-95 estimated periods

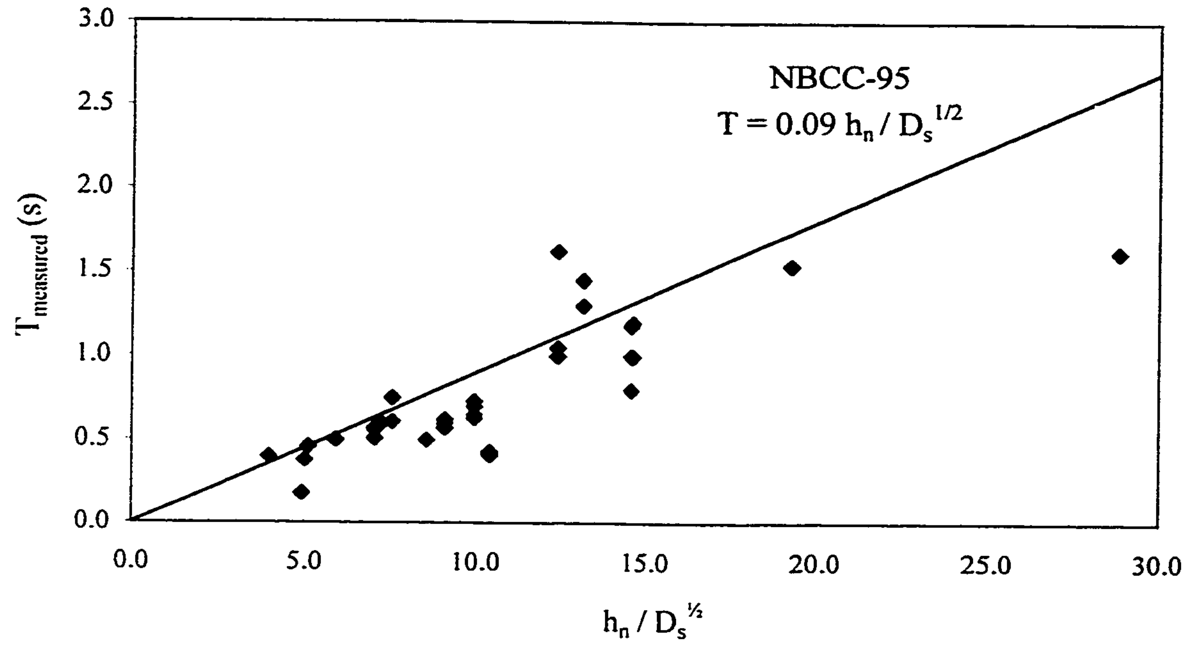


Fig. 2.4 Comparison between measured periods and the NBCC-95's estimated values when D_s is taken as the total length of all shear walls over the number or shear walls

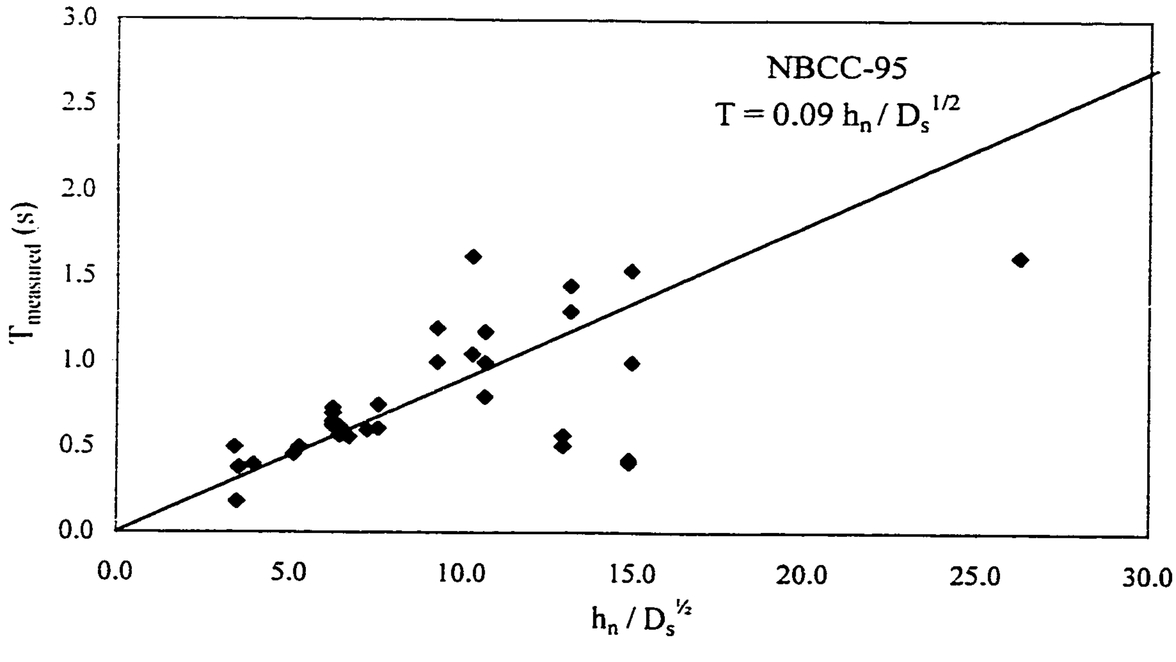


Fig. 2.5 Comparison between measured periods and the NBCC-95's estimated values when D_s is the sum of wall lengths in all frames over the number of frames containing these walls

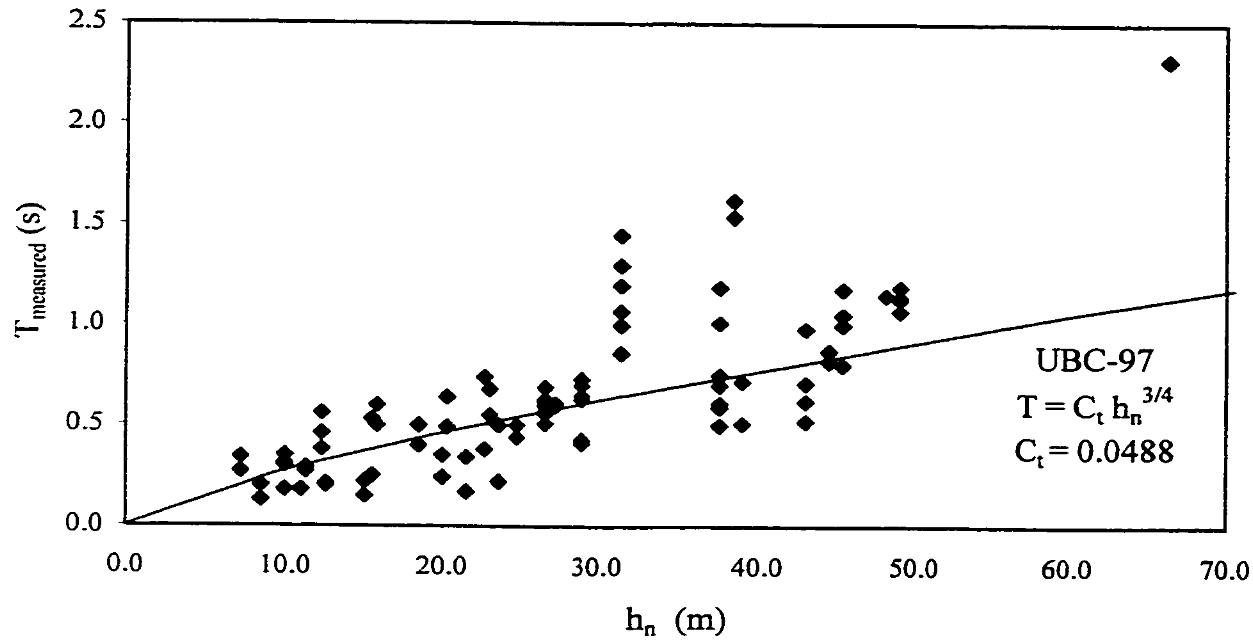


Fig. 2.6 Comparison between the measured periods and the UBC-97's estimated periods of shear wall buildings

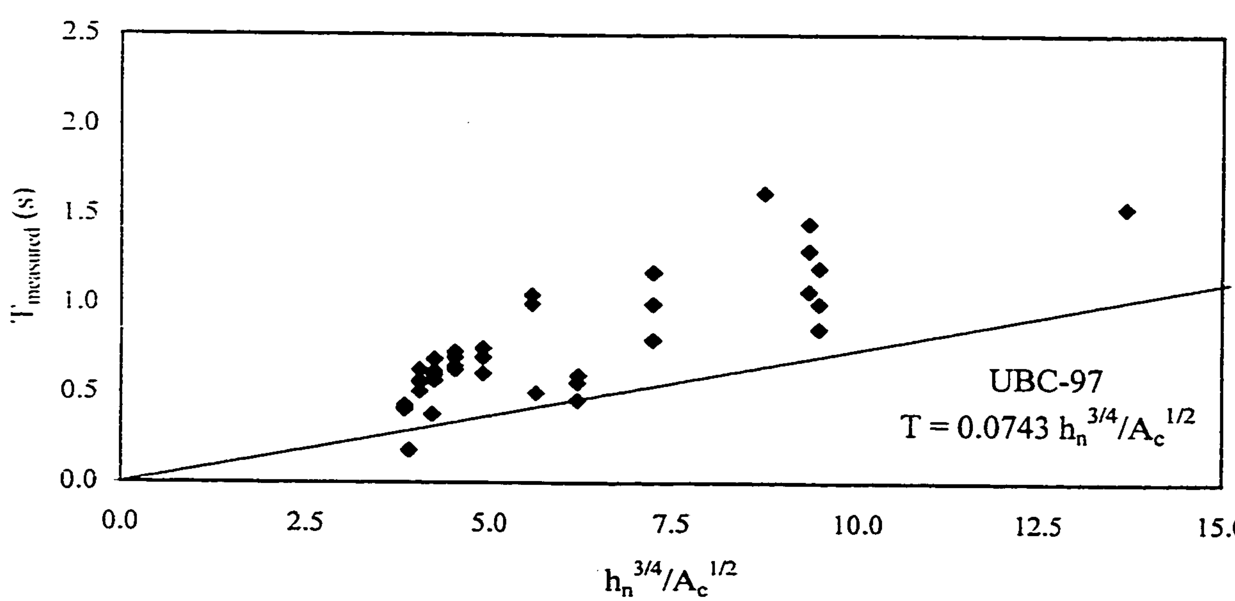


Fig. 2.7 Comparison between measured periods and the UBC-97 alternative formula to estimate the fundamental period of shear wall buildings

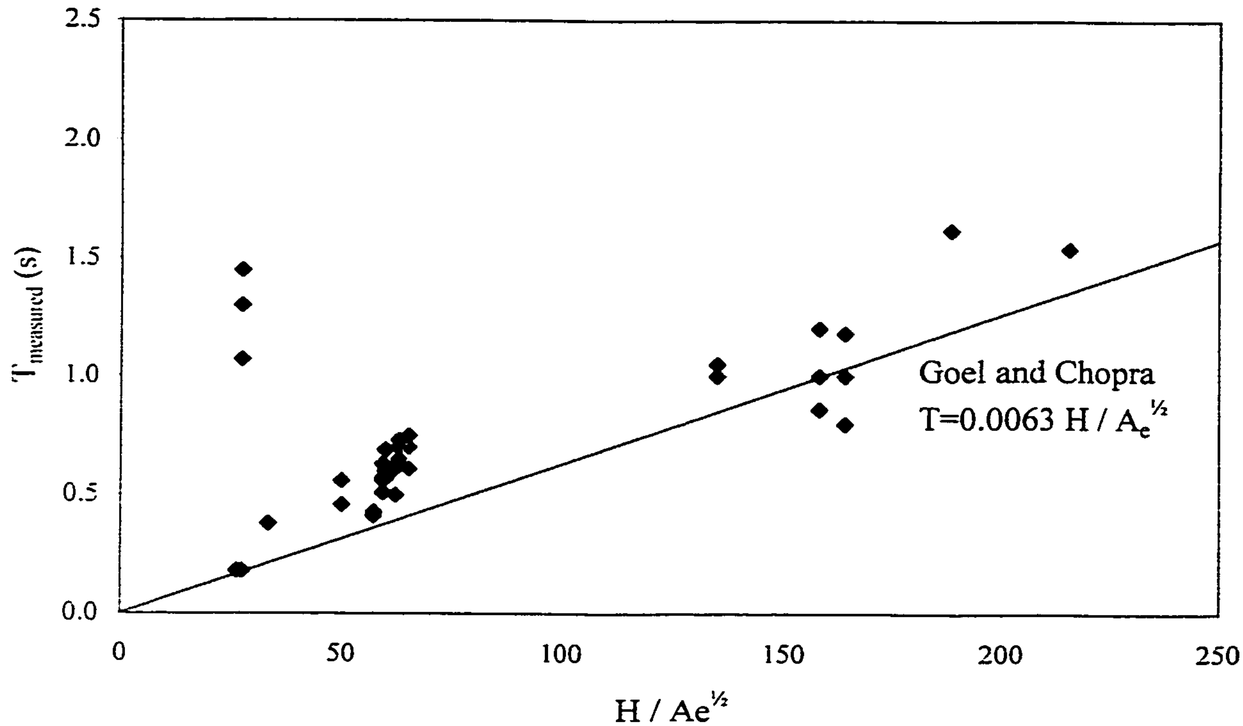


Fig. 2.8 Comparison between measured periods and Goel and Chopra's estimation for shear wall buildings

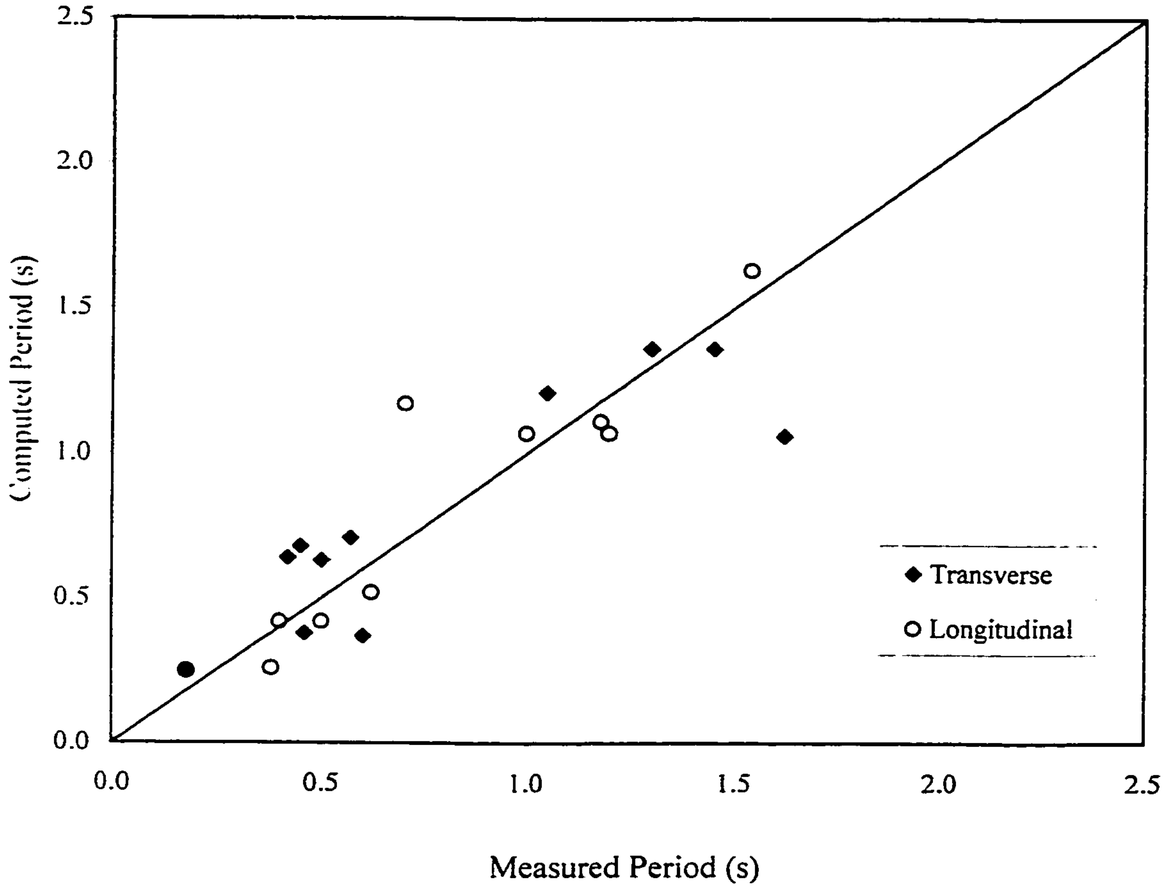


Fig. 2.9 Relationship between the measured and computed periods with SAP200 for selected shear wall buildings

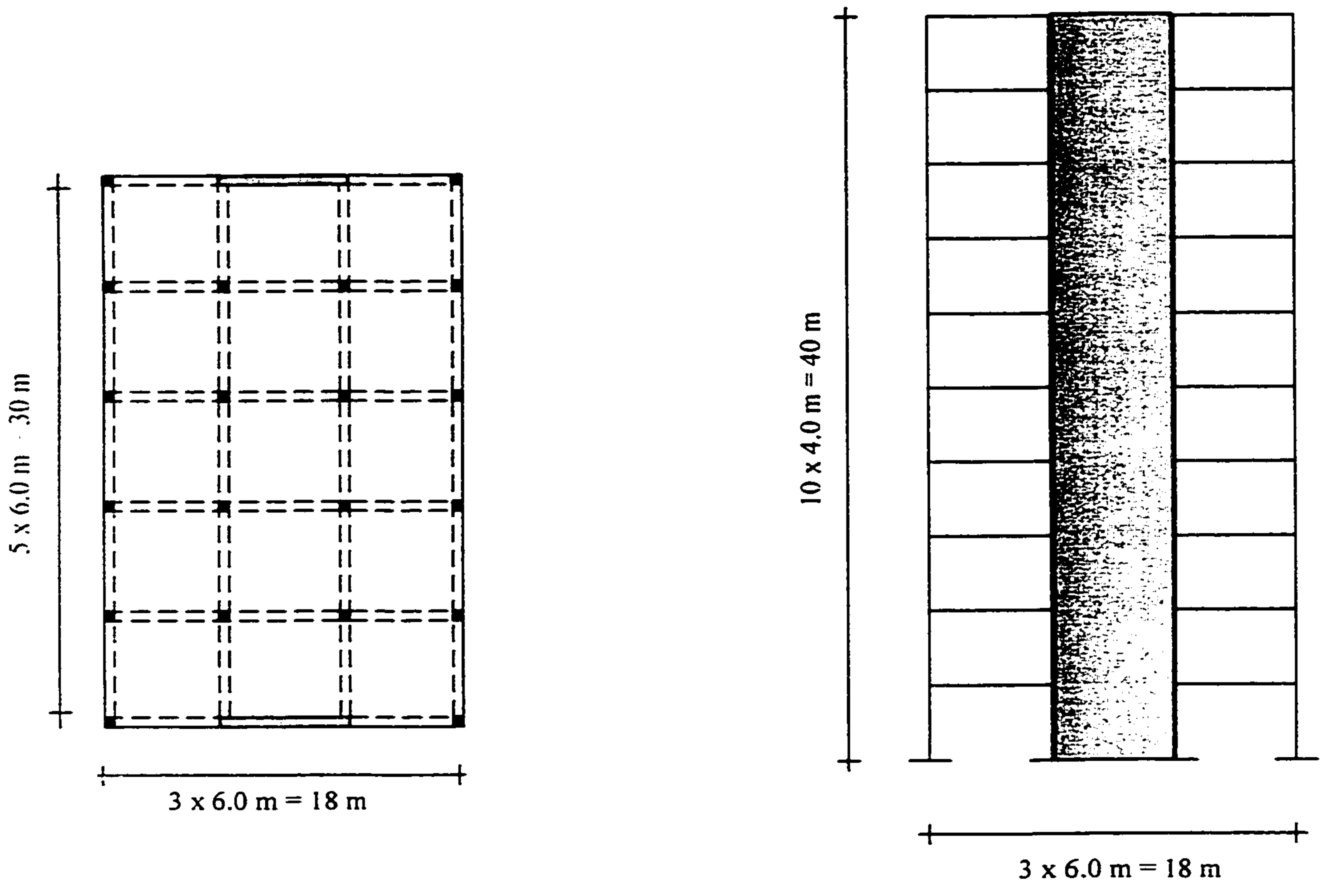


Fig. 2.10 Typical plan and elevation for 5-, 10- and 15-storey shear wall analytical models

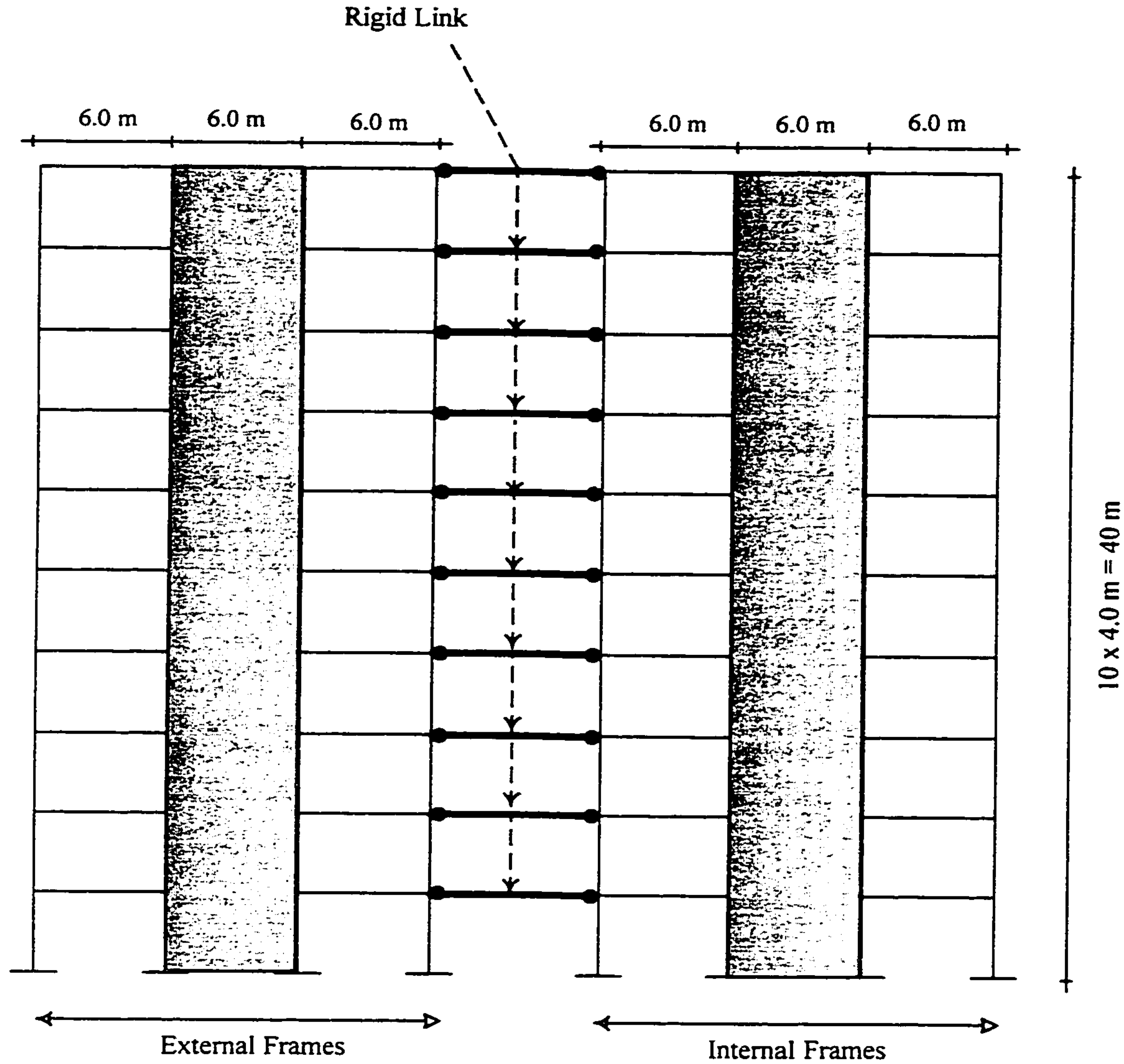


Fig. 2.11 Typical two-dimensional lumped frame used in analyses

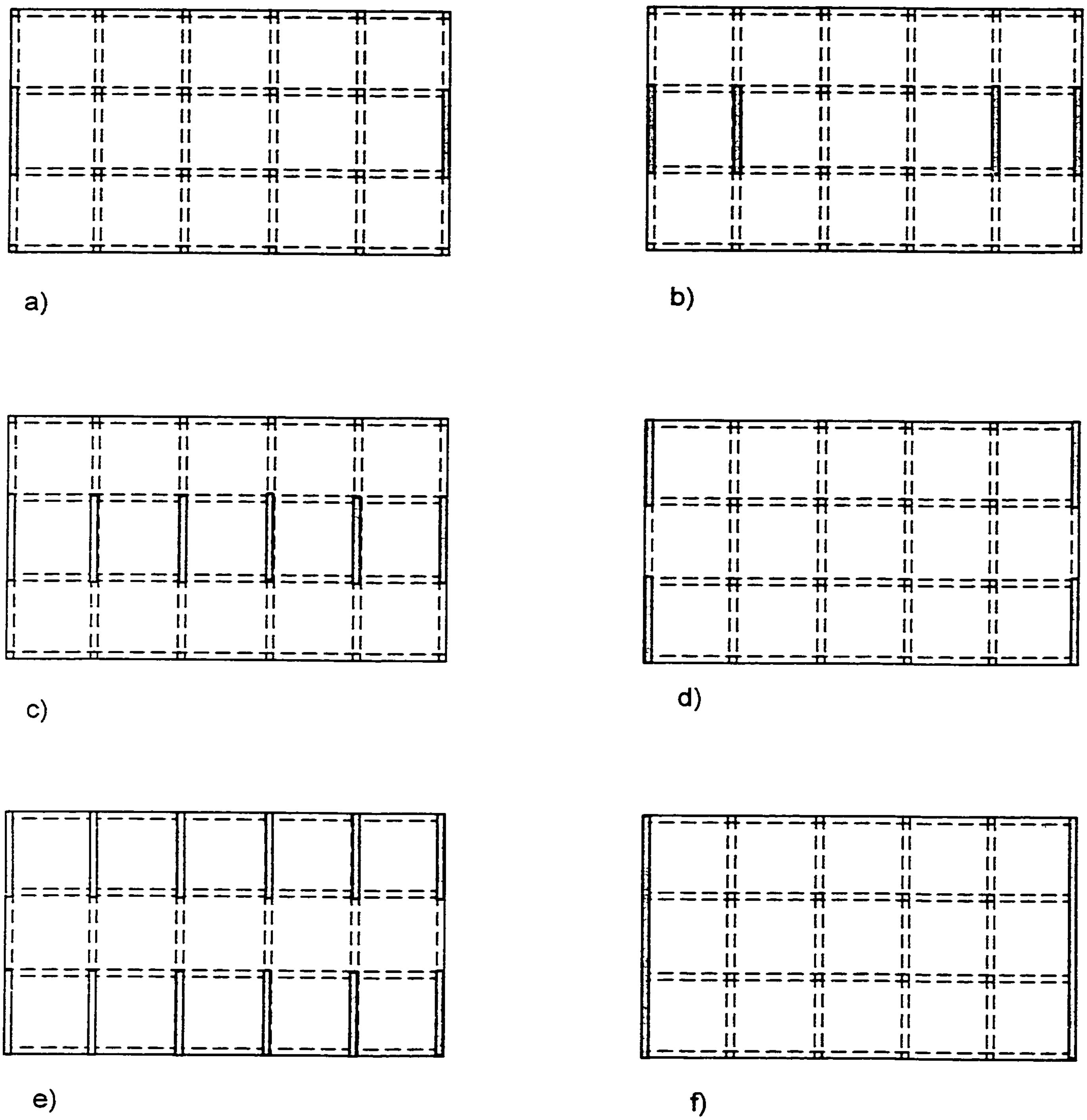


Fig. 2.12 Layout of shear walls in the analytical models

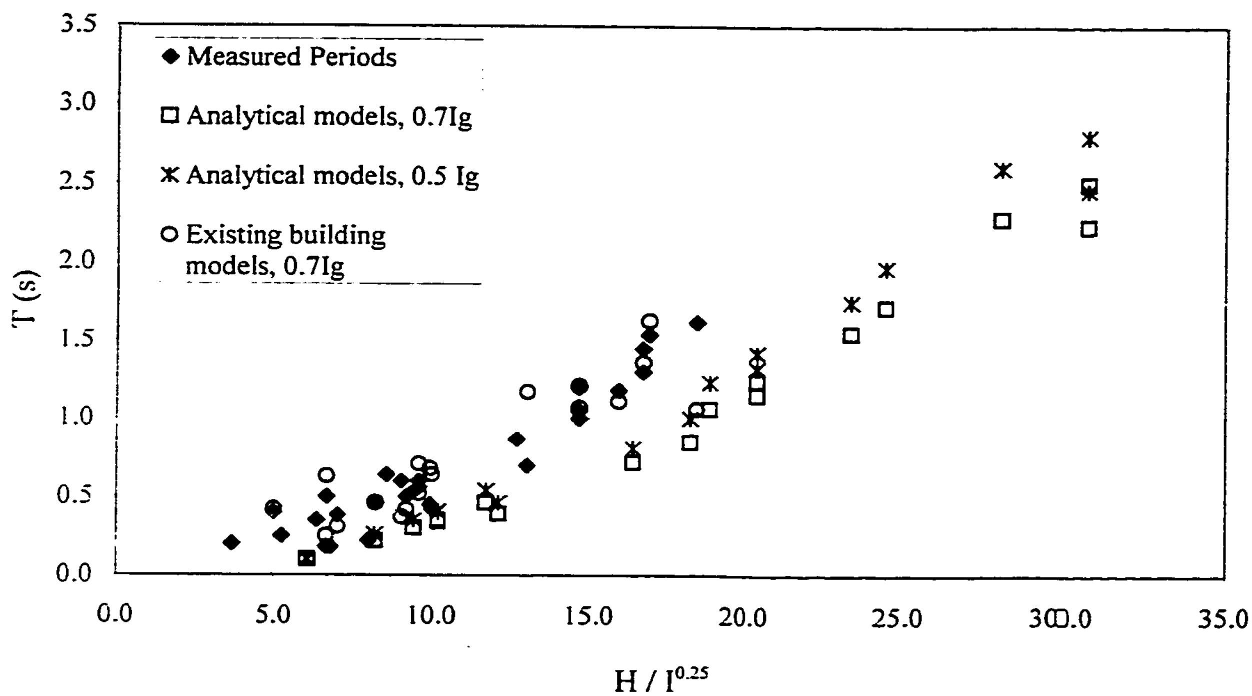


Fig. 2.13 Effect of the cracked section on the fundamental period of shear wall buildings.

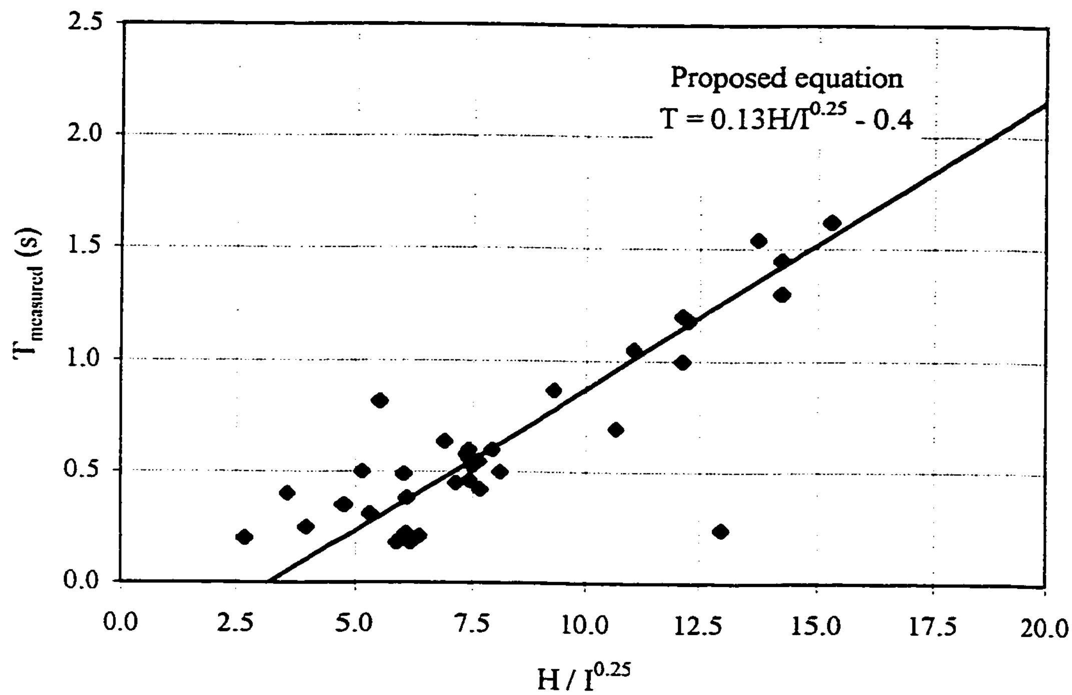


Figure 2.14 Correlation between the measured periods for selected shear wall buildings and the proposed formula

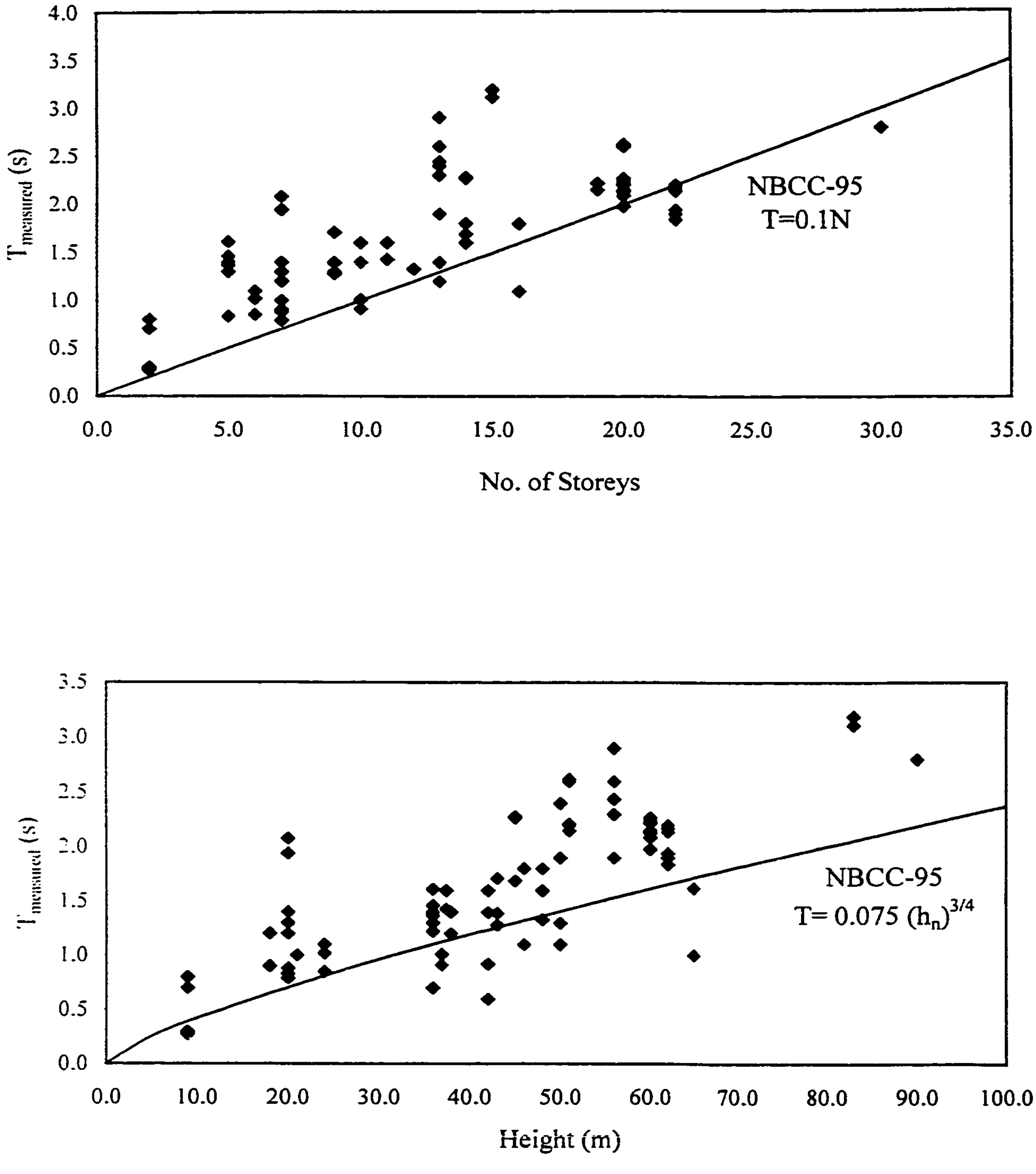


Fig. 3.1 Comparison between the measured periods and the NBCC-95 estimated periods for reinforced concrete moment-resisting frame buildings

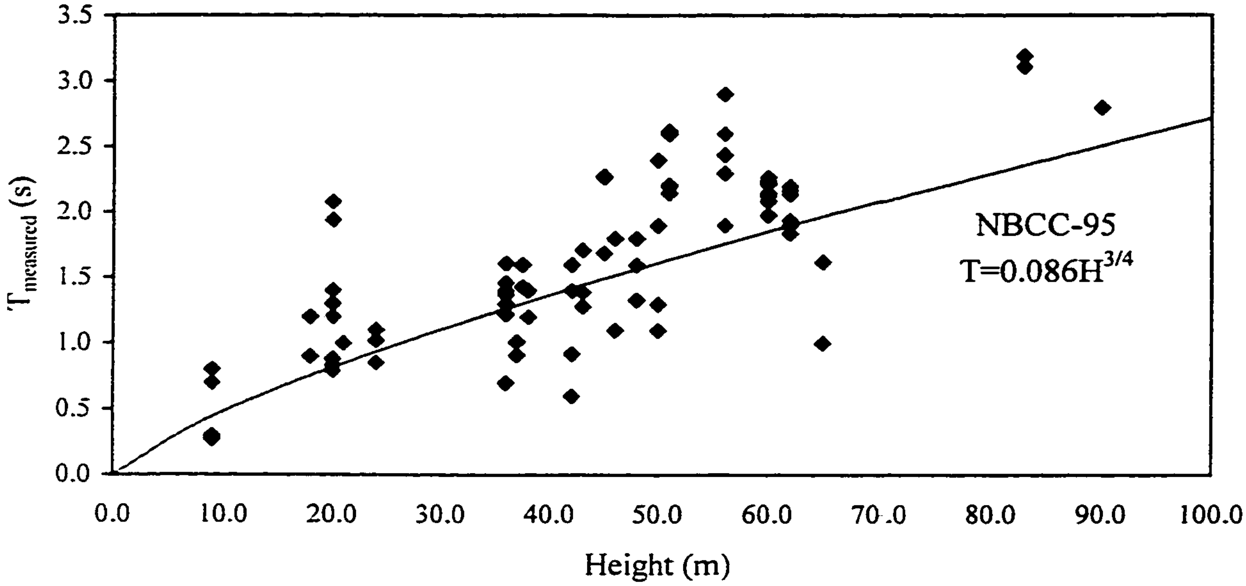


Fig. 3.2 Comparison between measured periods and the NBCC-95 estimated periods when C_t= 0.086, moment-resisting frame buildings

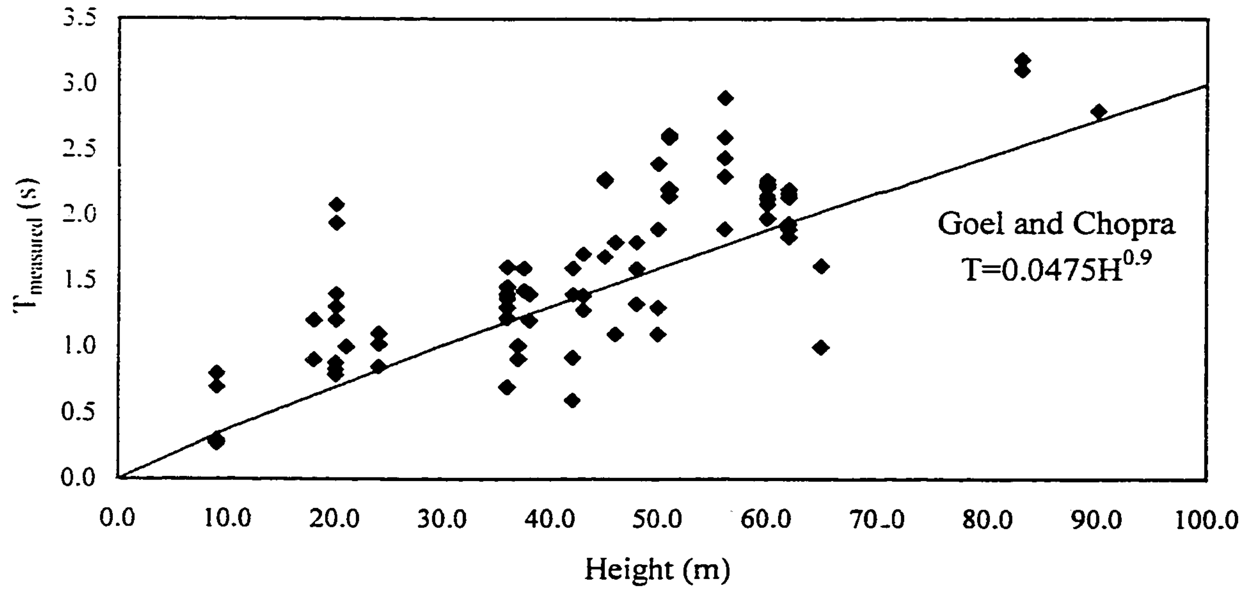


Fig. 3.3 Comparison between measured periods and Goel and Chopra's formula estimation for moment-resisting frame buildings

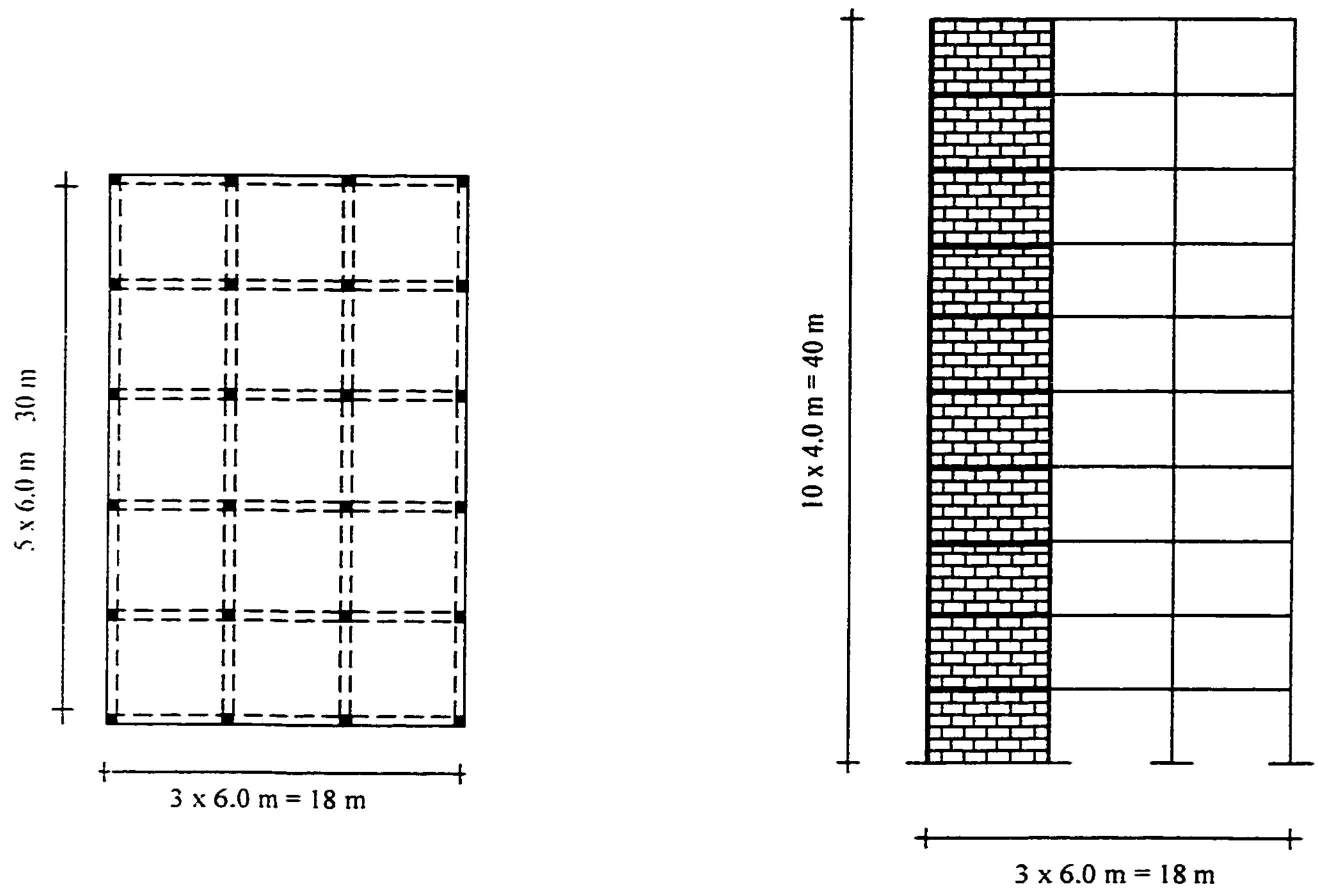


Fig. 3.4 Typical plan and elevation for 5-, 10- and 15-storey moment-resisting frame models

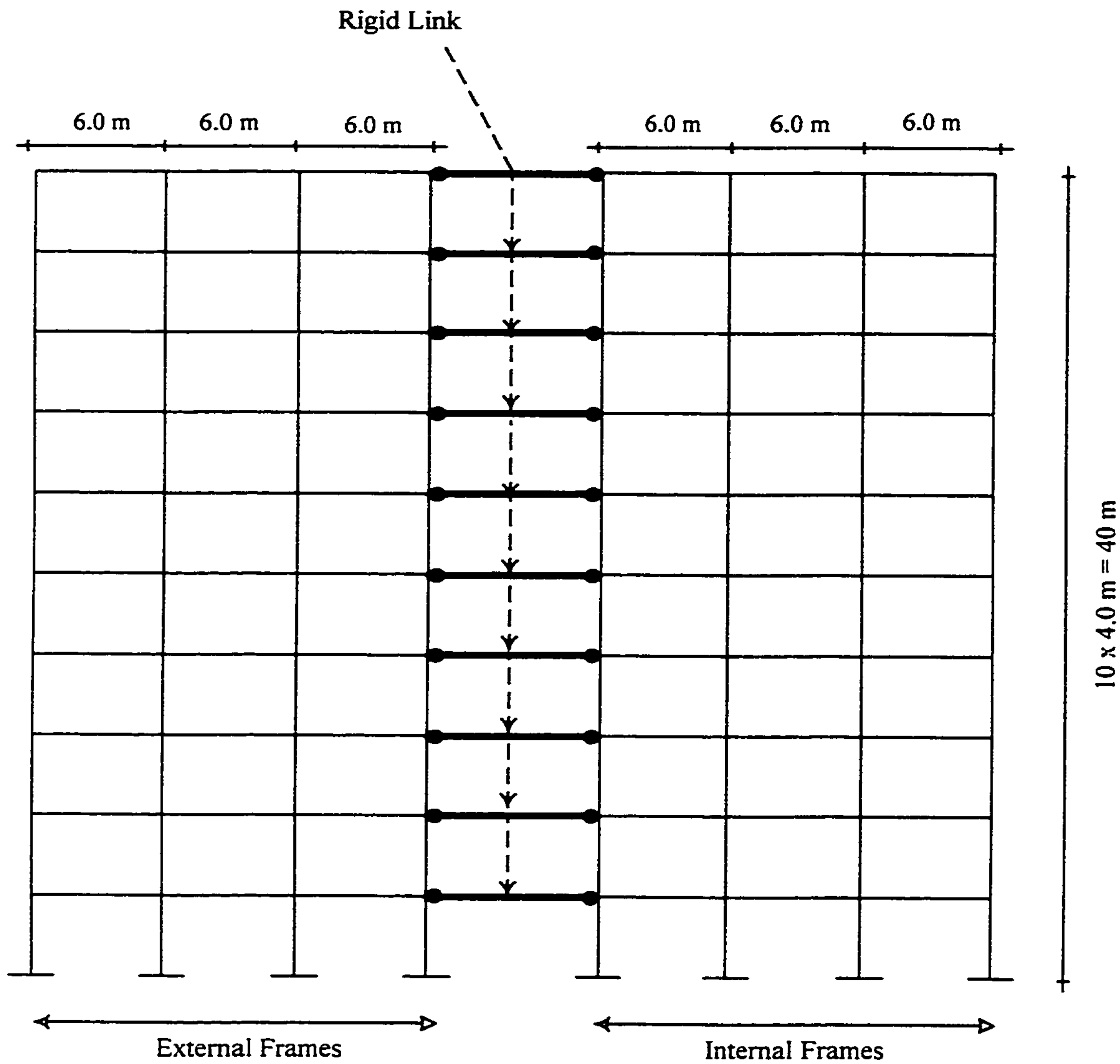


Fig. 3.5 Typical two-dimensional lumped frame used in analysis

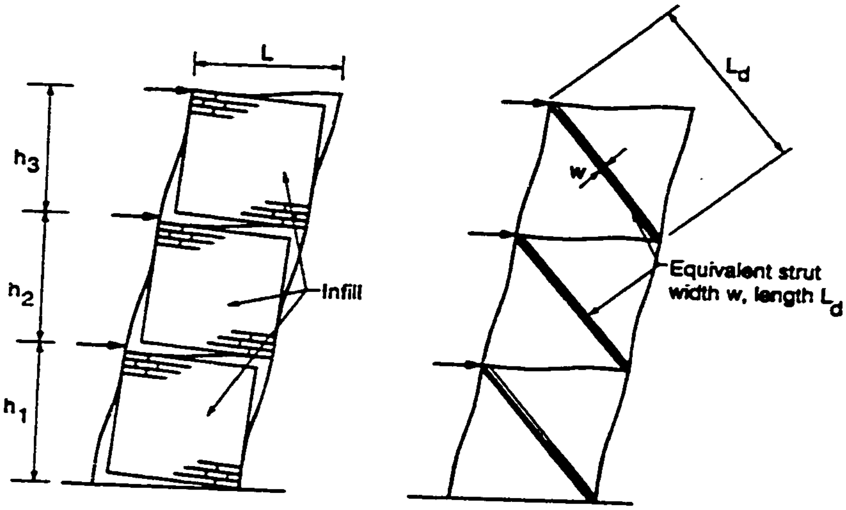


Fig. 3.6 Equivalent diagonal strut method for infill walls

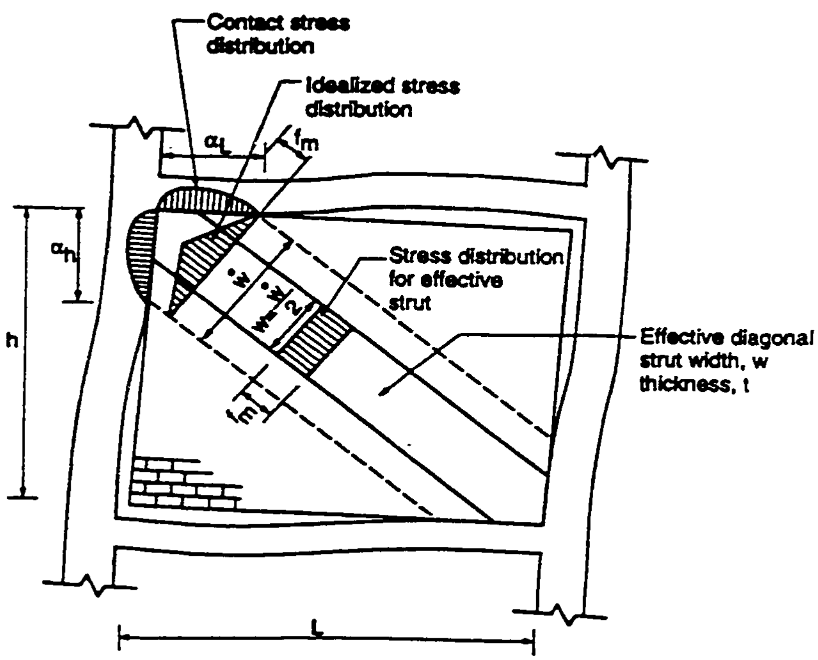


Fig. 3.7 Geometric properties of the equivalent diagonal strut

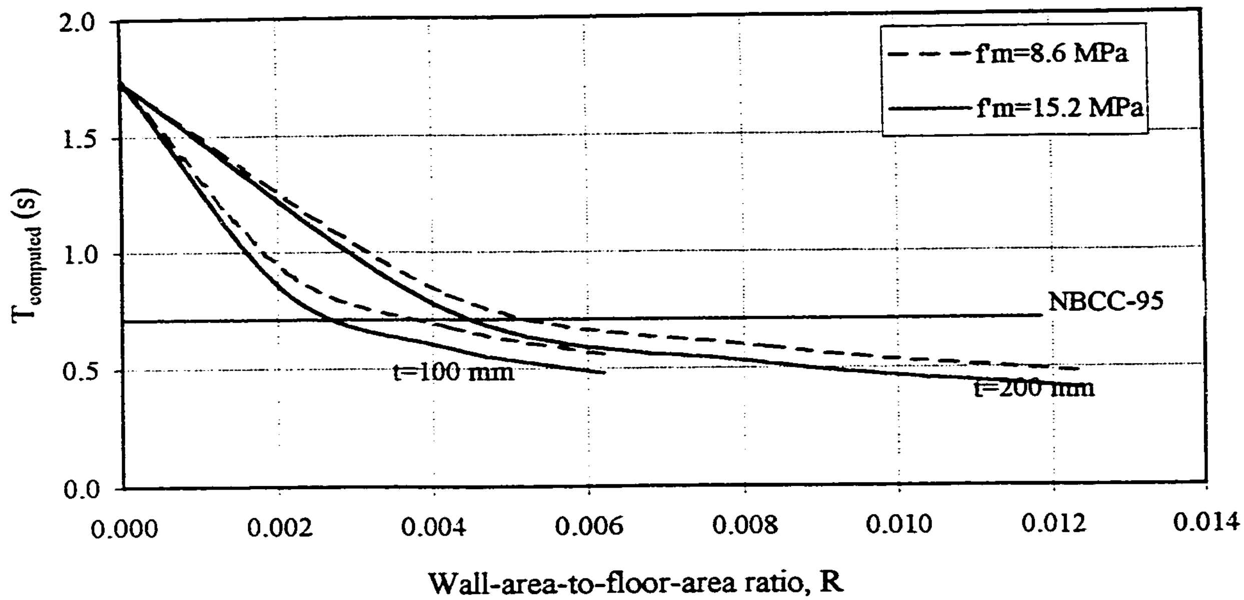


Fig. 3.8 Effect of masonry infill walls on the fundamental period of the 5-storey building

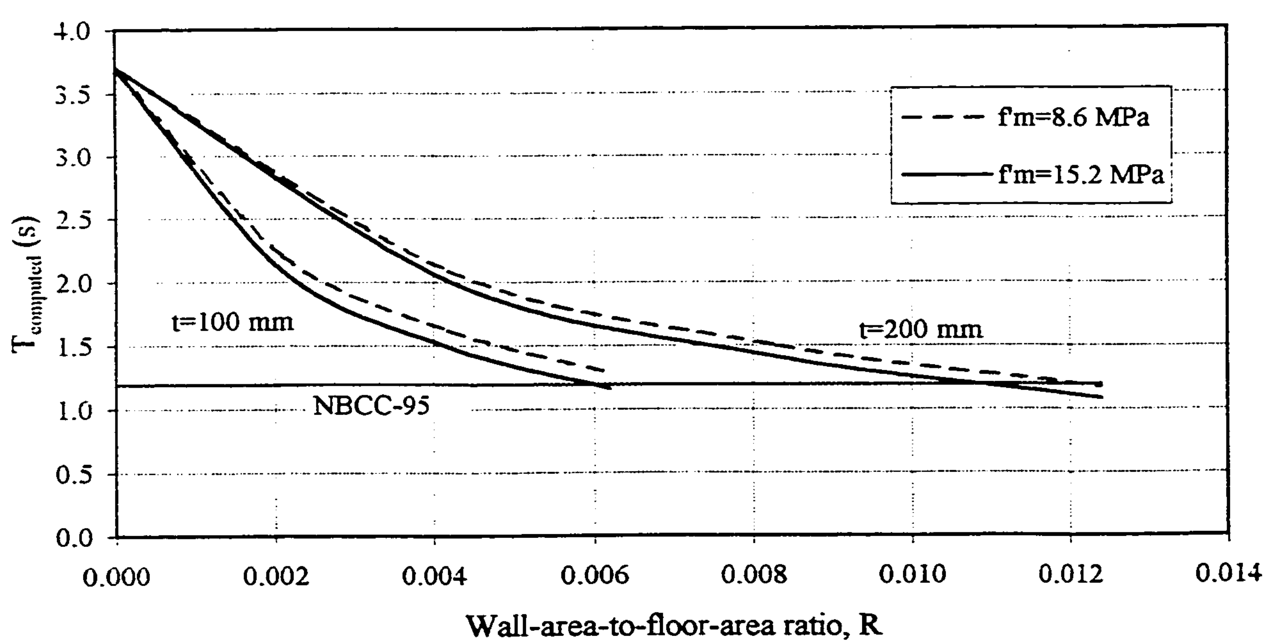


Fig. 3.9 Effect of masonry infill walls on the fundamental period of the 10-storey building

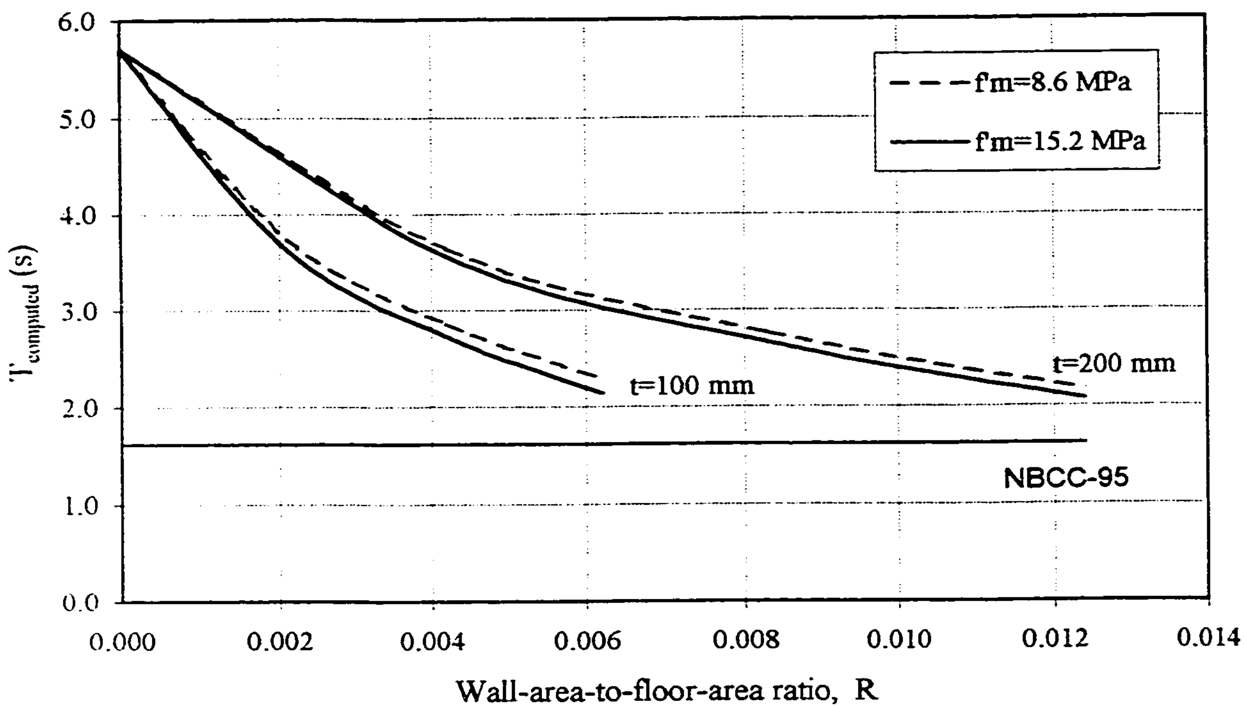


Fig. 3.10 Effect of masonry infill walls on the fundamental period of the 15-storey building

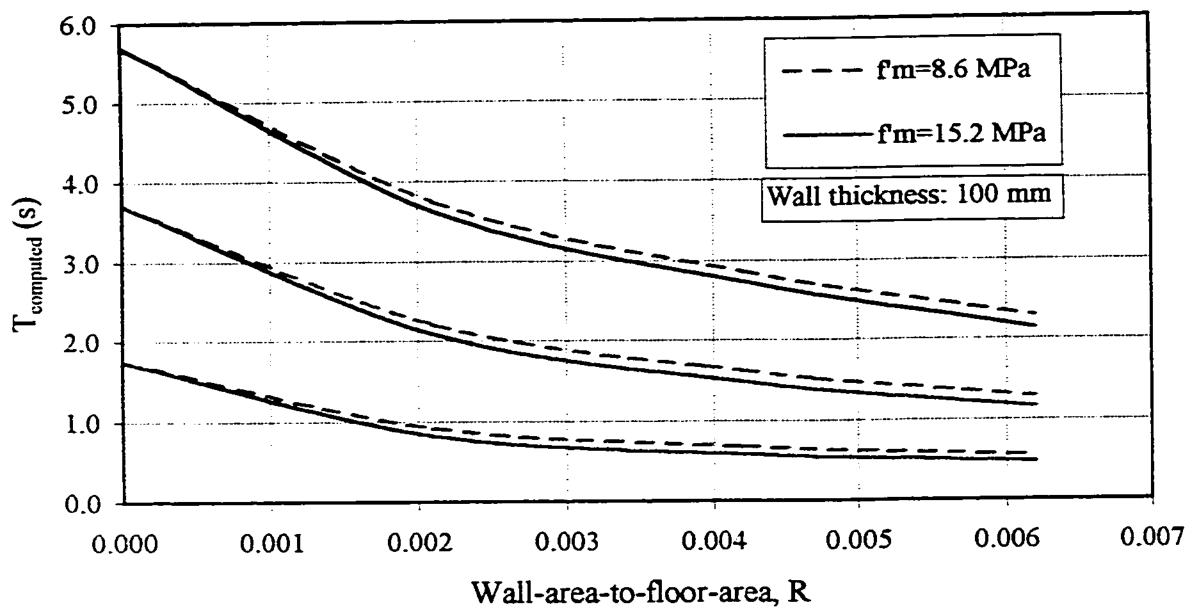


Fig. 3.11 Comparison between the fundamental periods for the 5-, 10- and 15-storey building

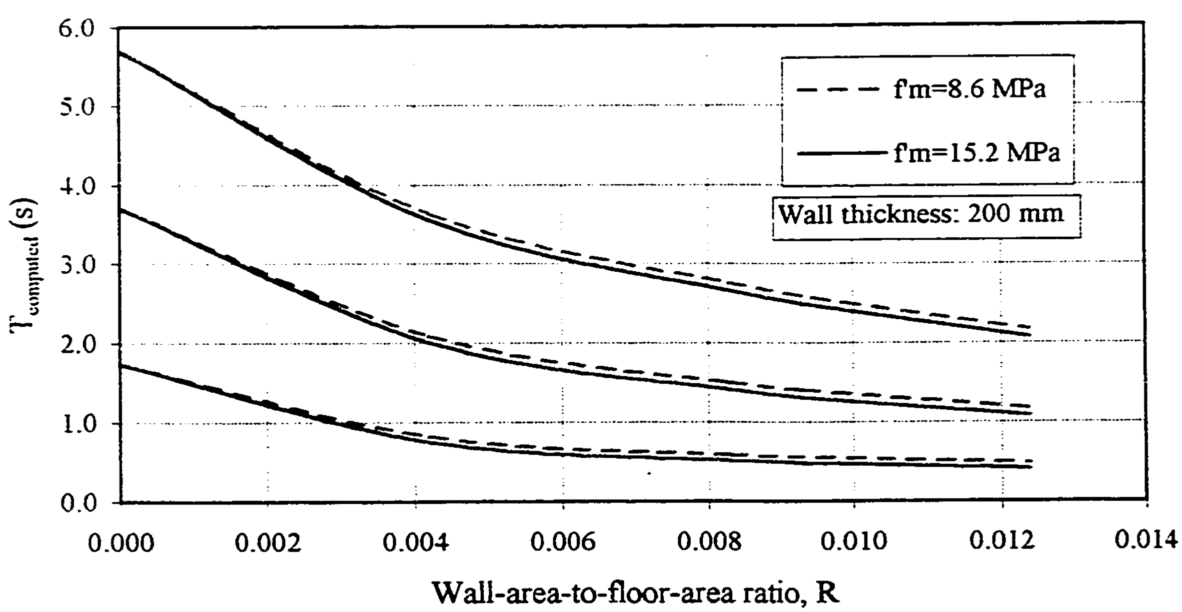


Fig. 3.12 Comparison between the fundamental periods for the 5-, 10- and 15-storey building

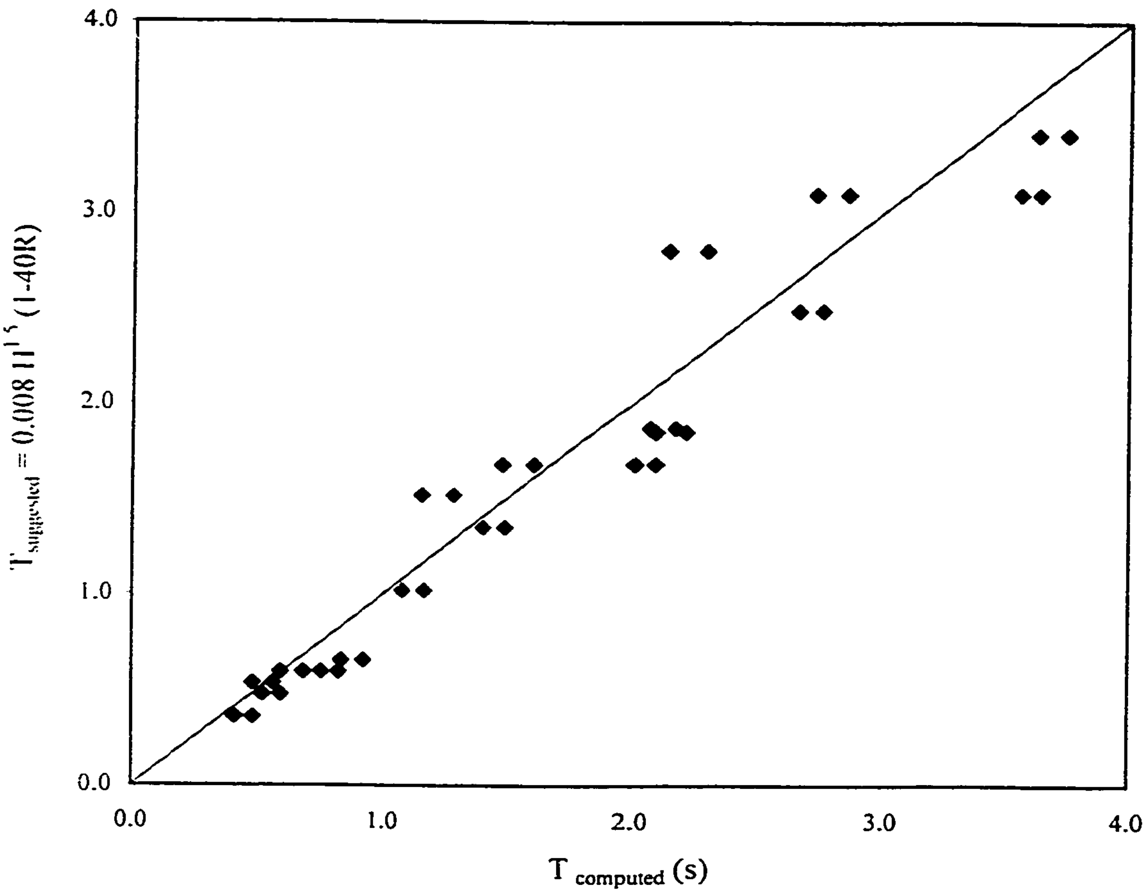


Fig. 3.13 Comparison between the suggested equation and the computed fundamental periods for moment-resisting frame buildings with infill walls.

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Appendix A-1

Burbank, 10-Story Residential Building C24601

Height 26.6 m
Plan area 1687 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	2.31	0.20	0.205 x 2 = 0.411	164.178	7.43
16	8.50	0.20	10.235 x 16 = 163.767		

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	5.40	0.20	2.624 x 2 = 5.249	164.307	7.43
4	10.50	0.20	19.294 x 4 = 77.175		
8	8.50	0.20	10.235 x 8 = 81.883		

A-1.1a Moment of inertia calculation, Burbank Building

Los Angeles, 8-Story Administration Building C24468

Height 38.5 m
Plan area 2957 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
23	0.90	0.25	0.015 x 23 = 0.349	40.908	15.22
1	3.97	0.25	1.304 x 1 = 1.304		
1	5.71	0.25	3.879 x 1 = 3.879		
1	3.65	0.25	1.013 x 1 = 1.013		
1	6.25	0.25	5.086 x 1 = 5.086		
1	4.40	0.25	1.775 x 1 = 1.775		
1	10.97	0.25	27.503 x 1 = 27.503		

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	1.52	0.25	0.073 x 1 = 0.073	32.064	16.18
1	11.40	0.25	30.866 x 1 = 30.866		
1	2.67	0.25	0.397 x 1 = 0.397		
1	1.30	0.25	0.046 x 1 = 0.046		
1	3.20	0.25	0.683 x 1 = 0.683		

A-1.2 Moment of inertia calculation, Los Angeles Building

Los Angeles, 10-Story Residential Building C24601

Height 45.3 m
Plan area 1687 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	8.80	0.20	11.358 x 2 = 22.716	282.967	11.04
16	8.50	0.20	10.235 x 16 = 163.767		
4	11.30	0.20	24.048 x 4 = 96.193		
2	2.06	0.20	0.146 x 2 = 0.291		

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
11	10.13	0.20	17.325 x 11 = 190.577	193.201	12.15
1	5.40	0.20	2.624 x 1 = 2.624		

A-1.3 Moment of inertia calculation, Los Angeles Building

Los Angeles, 6-Story Parking C24655

Height 18.5 m

Plan area 7231 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
6	9.90	0.35	28.300 x 6 = 169.802	169.802	5.12

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	21.80	0.40	345.341 x 2 = 690.682	690.682	3.61

A-1.4 Moment of inertia calculation, Los Angeles Building

Palm Desert, 4-Story Medical Building C12284

Height 15.8 m

Plan area 1003 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	3.00	0.25	0.563 x 2 = 1.125	13.657	8.22
2	6.70	0.25	6.266 x 2 = 12.532		

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H / I^{0.25}$
4	5.00	0.25	2.604 x 4 = 10.417	12.434	8.41
1	4.52	0.25	1.924 x 1 = 1.924		
1	0.43	0.25	0.002 x 1 = 0.002		
2	1.30	0.25	0.046 x 2 = 0.092		

A-1.5 Moment of inertia calculation, Palm Desert Building

Piedmont, 3-Story School Building C58344

Height 11 m
Plan area 754 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
4	5.00	0.30	3.125 x 4 = 12.500	12.500	5.85

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
4	5.00	0.30	3.125 x 4 = 12.500	12.500	5.85

A-1.6 Moment of inertia calculation, Piedmont Building

Pleasant Hill, 3-Story Commercial Building   C58348

Height   12.3 m
Plan area   937 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	5.80	0.23	3.740 x 2 = 7.479	7.479	7.44

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
4	6.10	0.23	4.350 x 4 = 17.402	17.402	6.02

A-1.7 Moment of inertia calculation, Pleasant Hill Building

San Bruno, 9-Story Office Building C58394

Height 31.5 m
Plan area 1498 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
5	5.80	0.30	4.878 x 5 = 24.389	24.389	14.17

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	12.20	0.30	45.396 x 1 = 45.396	47.697	11.99
3	2.54	0.30	0.410 x 3 = 1.229		
1	3.50	0.30	1.072 x 1 = 1.072		

A-1.8 Moment of inertia calculation, San Bruno Building

San Francisco, 6-Storey School Building C58479

Height 24 m

Plan area 1118 m^2

Transverse direction*

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	18.40	0.20	103.825 x 1 = 103.825	243.249	6.08
1	20.30	0.20	139.424 x 1 = 139.424		

* shear wall thickness was assumed

Longitudinal direction: N/A

A-1.9 Moment of inertia calculation, San Francisco Building

San Jose, 10-Storey Commercial Building   C57355

Height   37.5 m
Plan area   1589 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	25.00	0.30	390.625 x 2 = 781.250	781.250	7.09

Longitudinal direction: MRF

A-1.10 Moment of inertia calculation, San Jose Building

San Jose, 10-Storey Residential Building C57356

Height 29 m
Plan area 1249 m^2

Transverse direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	5.50	0.28	3.882 x 1 = 3.882	210.650	7.61
4	8.20	0.28	12.865 x 4 = 51.461		
13	8.00	0.28	11.947 x 13 = 155.307		

Longitudinal direction

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
6	7.30	0.28	9.077 x 6 = 54.462	174.664	7.98
5	10.10	0.28	24.040 x 5 = 120.202		

A-1.11 Moment of inertia calculation, San Jose Building

Saratoga, 1-Storey School Gym C58344

Height 10 m

Plan area 1723 m^2

Transverse direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	4.80	0.30	2.765 x 2 = 5.530	6.880	6.17
16	1.50	0.30	0.084 x 16 = 1.350		

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
4	4.80	0.30	2.765 x 4 = 11.059	12.409	5.33
16	1.50	0.30	0.084 x 16 = 1.350		

A-1.12 Moment of inertia calculation, Saratoga Building

Watsonville, 4-Storey Commercial Building C47459

Height 20 m
Plan area 484 m^2

*Transverse direction**

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	7.00	0.35	10.004 x 1 = 10.004	307.906	4.77
3	3.30	0.35	1.048 x 3 = 3.144		
1	21.40	0.35	285.843 x 1 = 285.843		
1	4.10	0.25	1.436 x 1 = 1.436		
1	7.00	0.25	7.146 x 1 = 7.146		
4	1.10	0.25	0.028 x 4 = 0.111		
1	2.20	0.25	0.222 x 1 = 0.222		

* shear wall thickness was assumed

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
1	3.30	0.25	0.749 x 1 = 0.749	5.597	13.00
4	2.60	0.25	0.366 x 4 = 1.465		
4	1.50	0.25	0.070 x 4 = 0.281		
3	1.10	0.25	0.028 x 3 = 0.083		
1	3.00	0.25	0.563 x 1 = 0.563		
3	3.40	0.25	0.819 x 3 = 2.457		

* shear wall thickness was assumed

A-1.13 Moment of inertia calculation, Watsonville Building

Newport Beach, 11-Storey Hospital C13589

Height 44.5 m

Plan area 1086 m^2

Transverse direction*

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	23.30	0.25	263.528 x 2 = 527.056	527.056	9.29

* shear wall thickness was assumed

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	46.60	0.25	2108.223 x 2 = 4216.446	4216.446	5.52

* shear wall thickness was assumed

A-1.14 Moment of inertia calculation, New Port Beach Building

San Bernardino, 5-Storey Library C23285

Height 10 m

Plan area 1723 m^2

Transverse direction*

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	4.80	0.30	2.765 x 2 = 5.530	6.880	6.17
16	1.50	0.30	0.084 x 16 = 1.350		

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
4	4.80	0.30	2.765 x 4 = 11.059	12.409	5.33
16	1.50	0.30	0.084 x 16 = 1.350		

* shear wall thickness was assumed

A-1.15 Moment of Inertia calculation, San Bernardino Building

San Bernardino, 6-Storey Hotel C23287

Height 15.5 m
Plan area 918 m^2

Transverse direction*

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	6.50	0.20	4.577 x 2 = 9.154	214.793	4.05
26	7.80	0.20	7.909 x 26 = 205.639		

* shear wall thickness was assumed

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
14	4.24	0.20	1.270 x 14 = 17.786	17.786	7.55

* shear wall thickness was assumed

A-1.16 Moment of inertia calculation, San Bernardino Building

Lancaster, 3-Storey Office Building C24517

Height 12.6 m

Plan area 827 m^2

Transverse direction*

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
3	21.20	0.20	158.802 x 3 = 476.406	476.406	2.70

* shear wall thickness was assumed

Longitudinal direction *

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
2	6.67	0.20	4.946 x 2 = 9.891	15.287	6.37
2	5.45	0.20	2.698 x 2 = 5.396		

* shear wall thickness was assumed

A-1.17 Moment of inertia calculation, Lancaster Building

Whittier, 8-Storey Hotel C14606

Height 23 m

Plan area 1117 m^2

*Transverse direction**

Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H/I^{0.25}$
6	6.67	0.25	6.182 x 6 = 37.093	82.050	7.64
12	5.45	0.25	3.372 x 12 = 40.470		
1	3.00	0.25	0.563 x 1 = 0.563		
1	4.80	0.25	2.304 x 1 = 2.304		
1	4.14	0.25	1.478 x 1 = 1.478		
1	1.90	0.25	0.143 x 1 = 0.143		

* shear wall thickness was assumed

*Longitudinal direction **

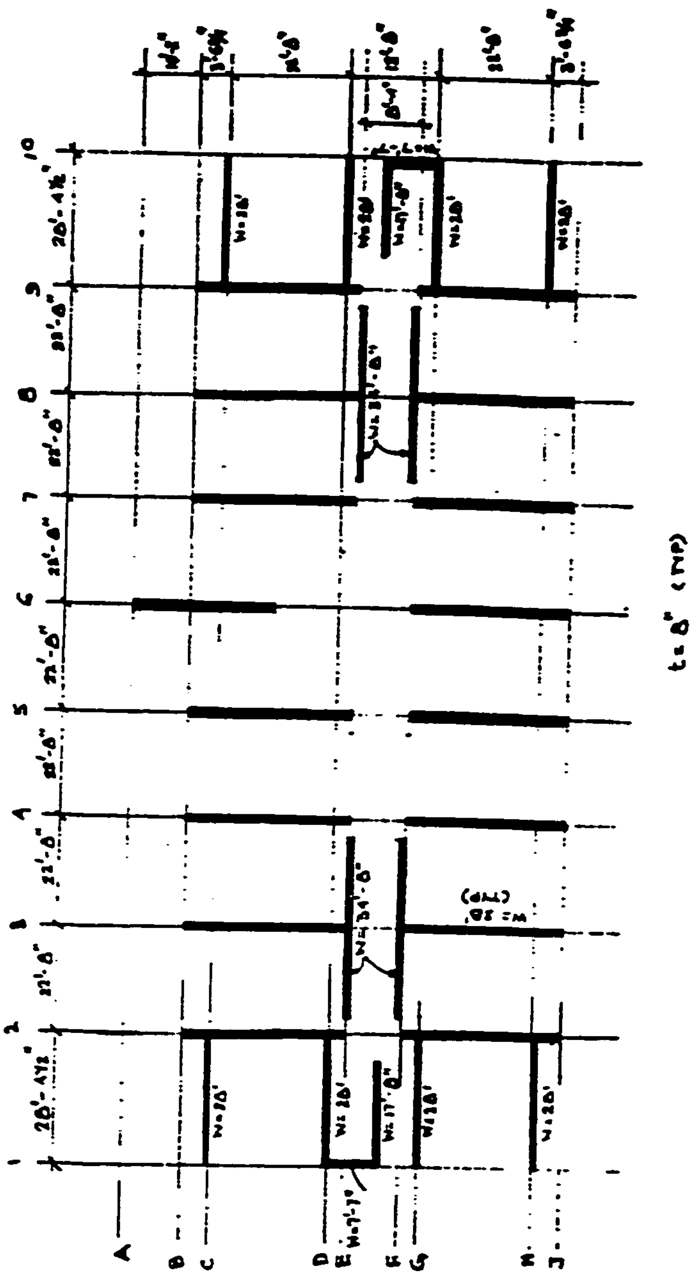
Number of walls	Length (m)	Thickness (m)	Moment of inertia I_{wall} (m^4)	$I = \Sigma I_{wall}$	$H / I^{0.25}$
21	5.80	0.25	4.065 x 21 = 85.362	95.118	7.36
9	2.90	0.25	0.508 x 9 = 4.573		
3	4.14	0.25	1.478 x 3 = 4.435		
1	3.30	0.25	0.749 x 1 = 0.749		

* shear wall thickness was assumed

A-1.18 Moment of inertia calculation, Whittier Building

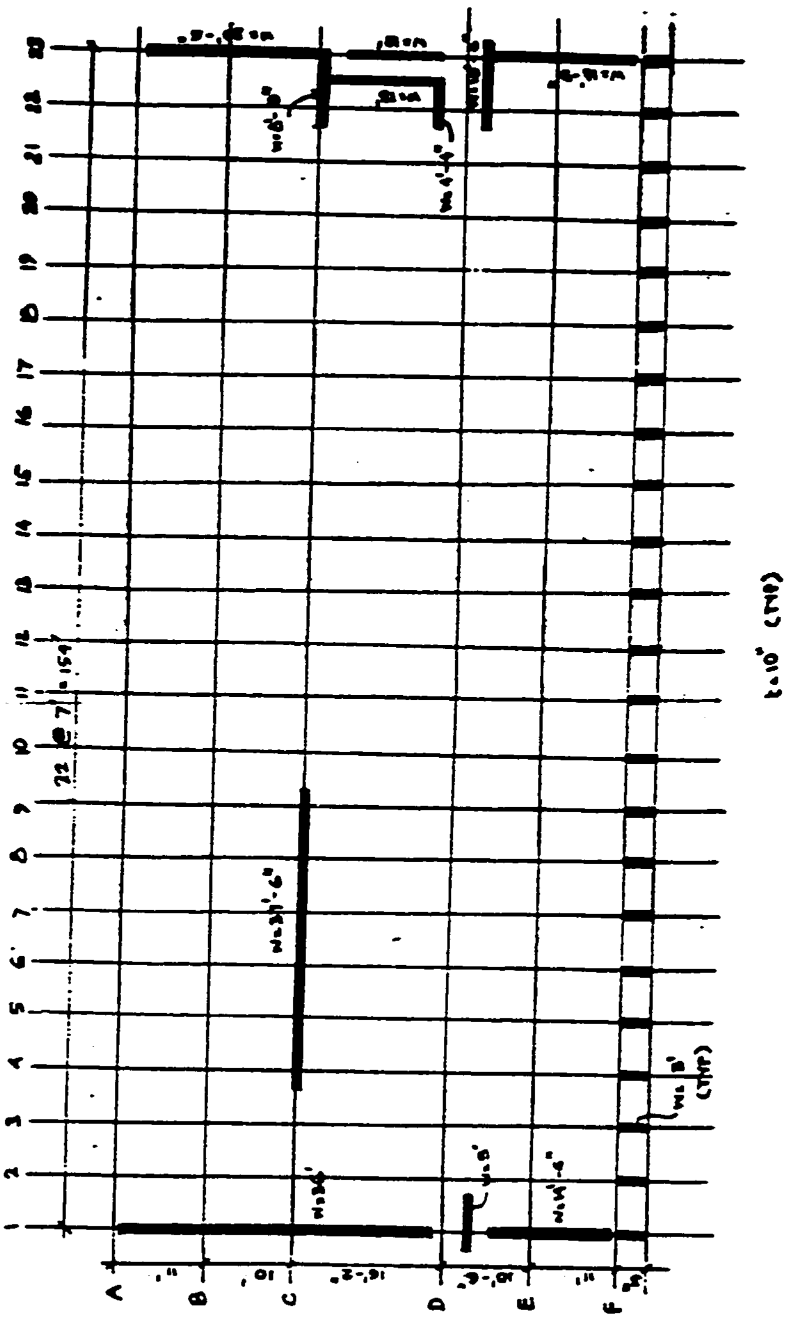
Burbank, 10-Story Residential Building

CSMIP Station No. 24601



A-1.1 Sketch of shear walls, Burbank Building

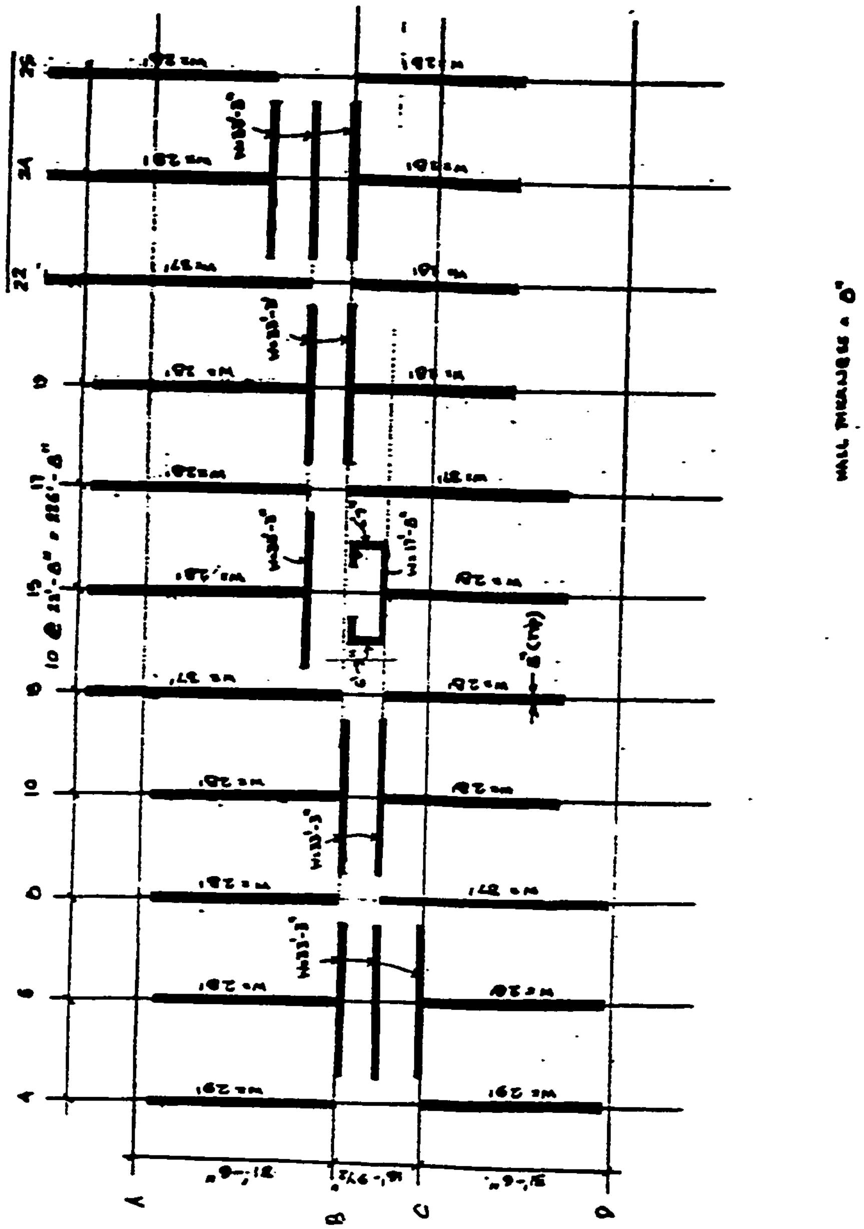
Los Angeles, 8-Story Administration Building
CSMIP Station No. 24468



A-1.2 Sketch of shear walls, Los Angeles Building

Los Angeles, 10-Story Residential Building

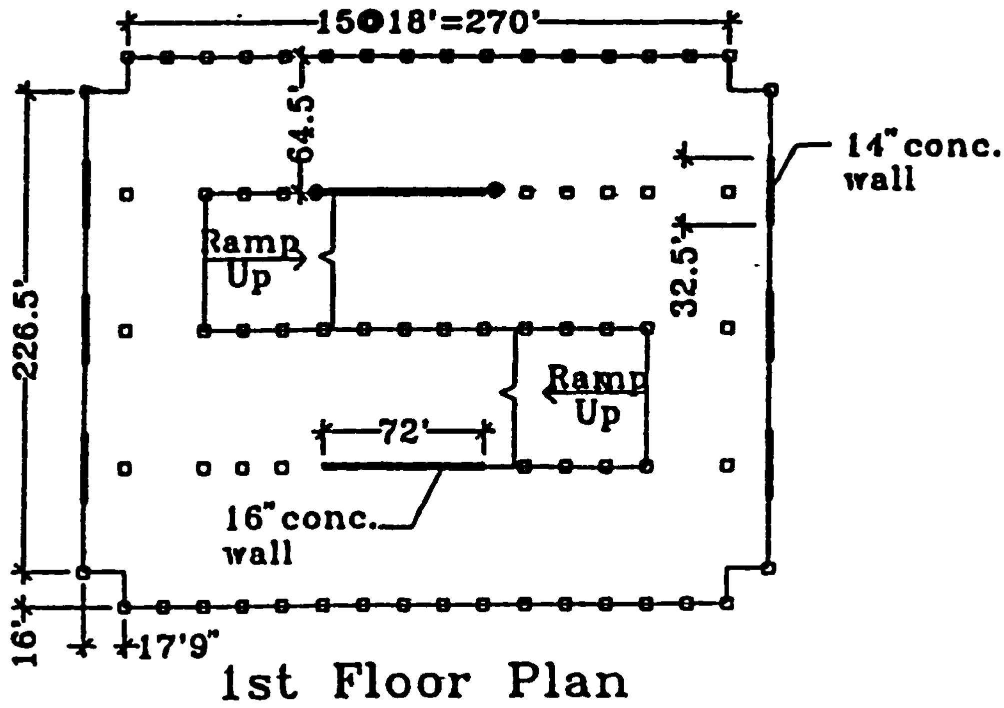
CSMIP Station No. 24601



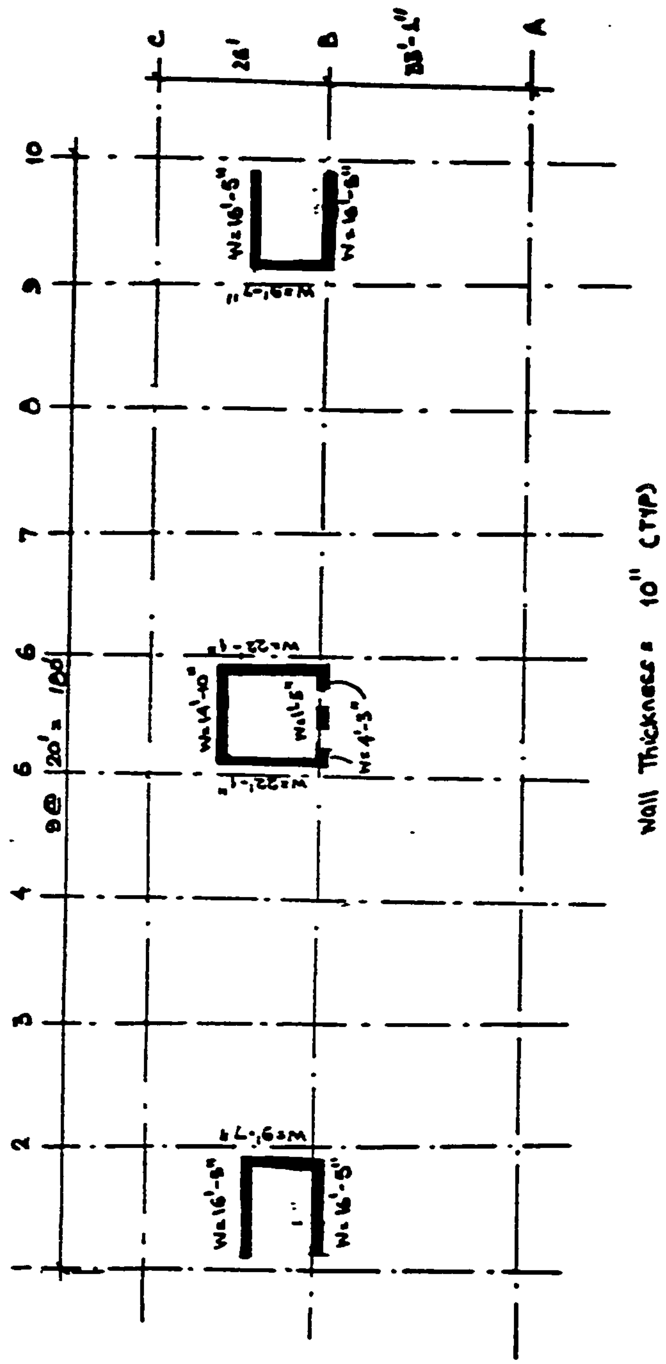
A-1.3 Sketch of shear walls, Los Angeles Building

Los Angeles, 6-Story Parking Building

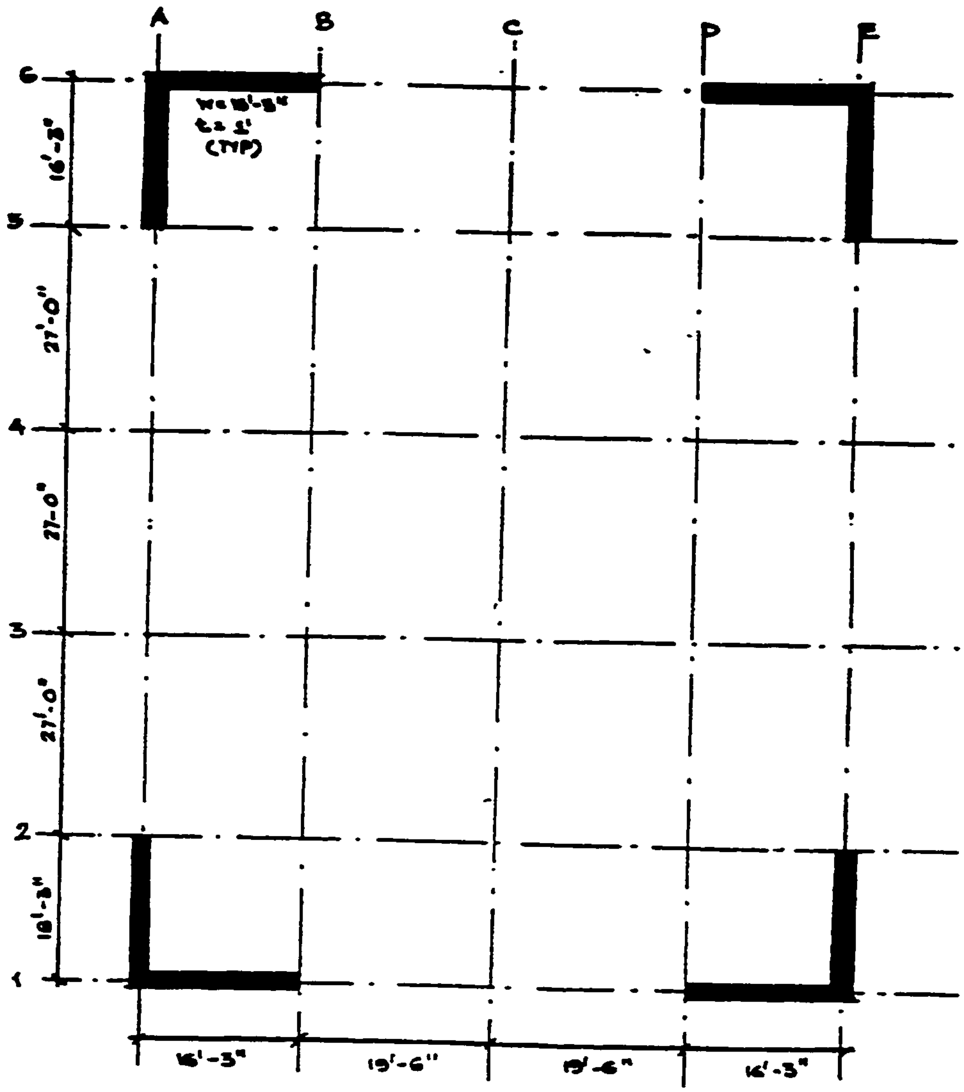
CSMIP Station No. 24655



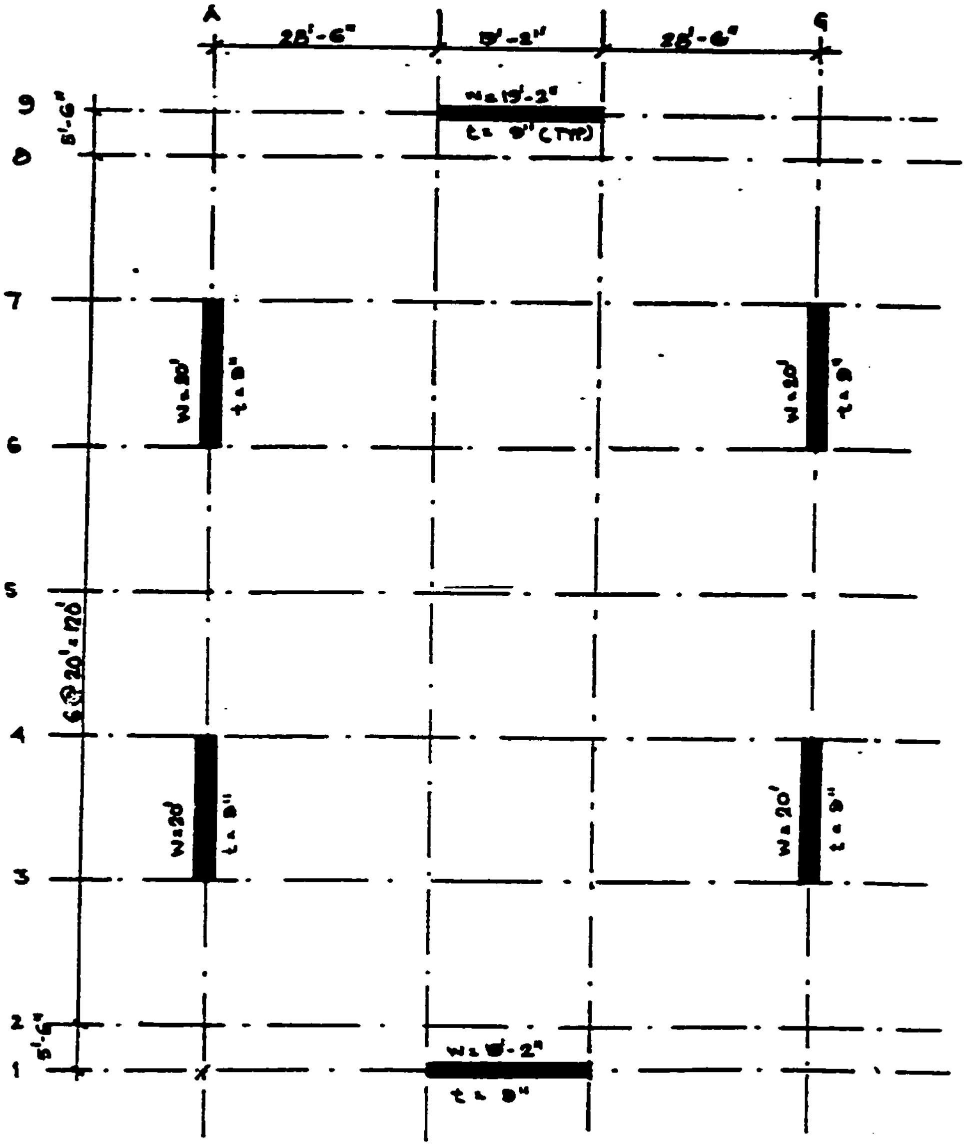
A-1.4 Sketch of shear walls, Los Angeles Building



A-1.5 Sketch of shear walls, Palm Desert Building



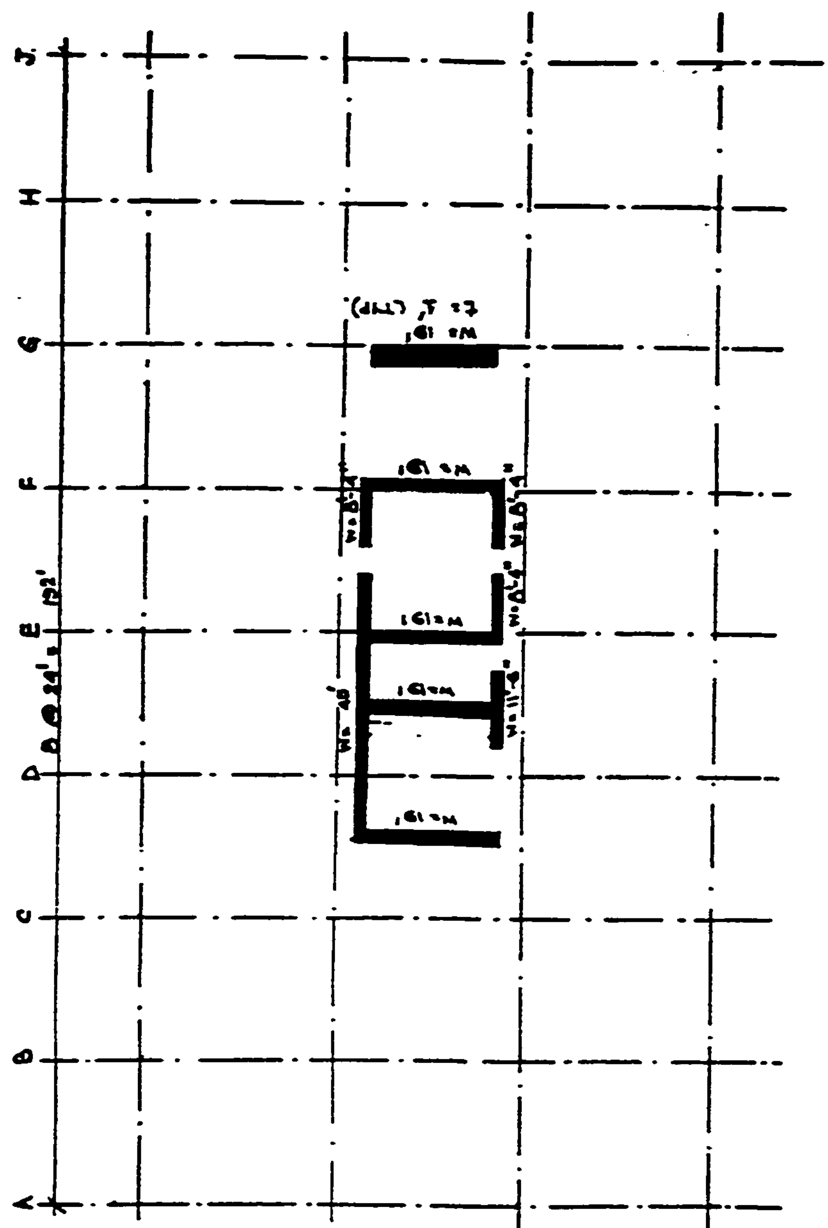
A-1.6 Sketch of shear walls, Piedmont Building



A-1.7 Sketch of shear walls, Pleasant Hill Building

San Bruno, 9-Story Office Building

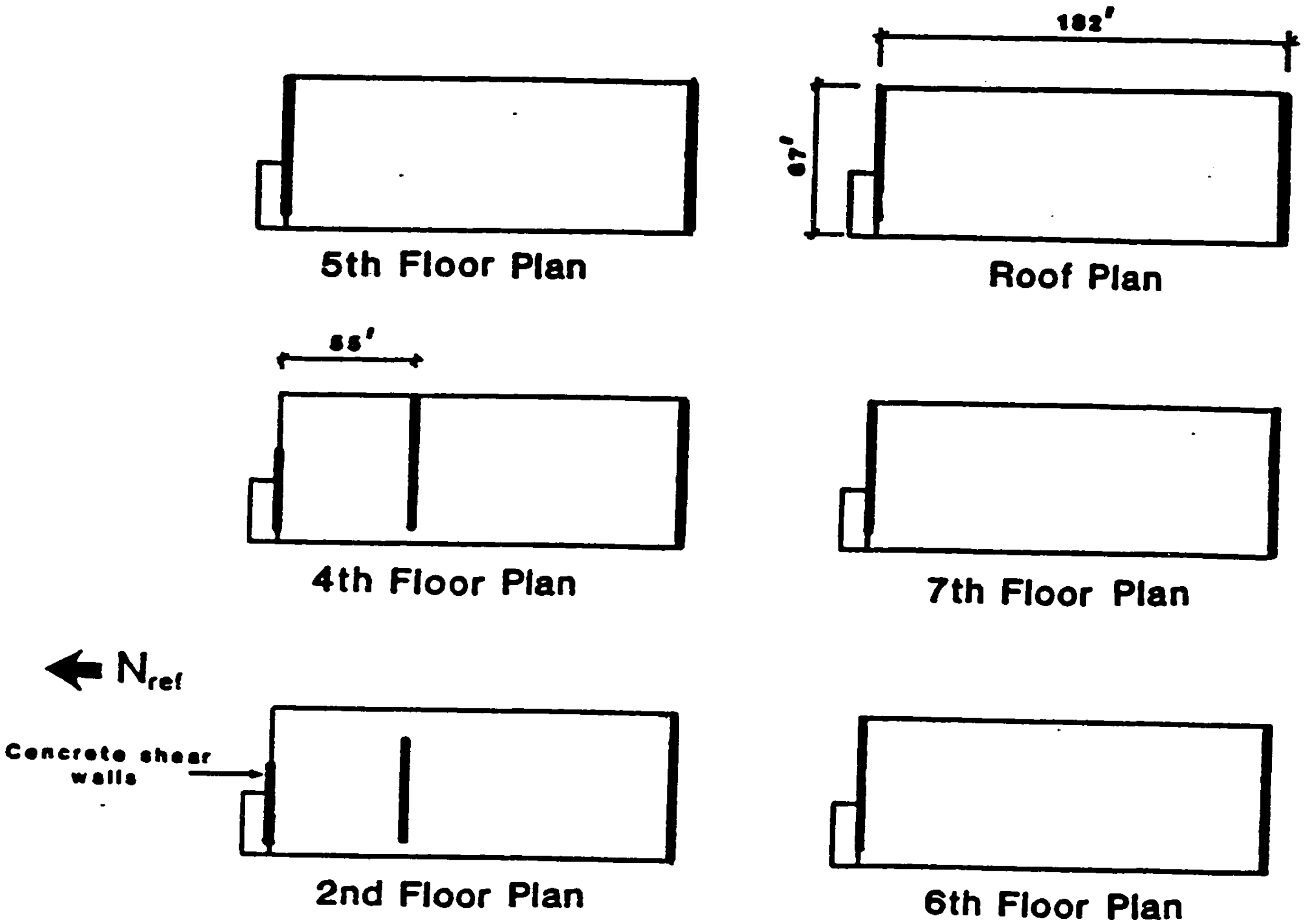
CSMIP Station No. 58394



A-1.8 Sketch of shear walls, San Bruno Building

San Francisco, 6-Story School Building

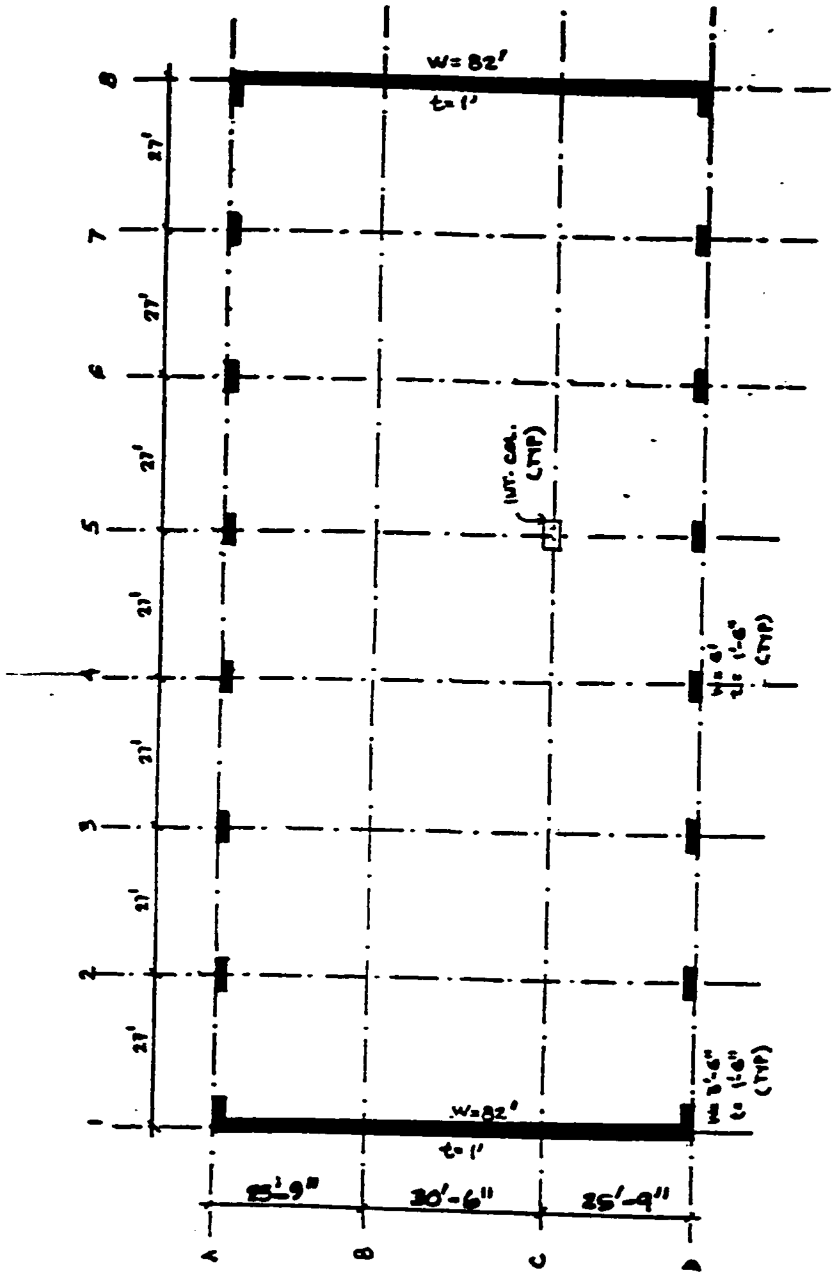
CSMIP Station No. 58479



A-1.9 Sketch of shear walls, San Francisco Building

San Jose, 10-Story Commercial Building

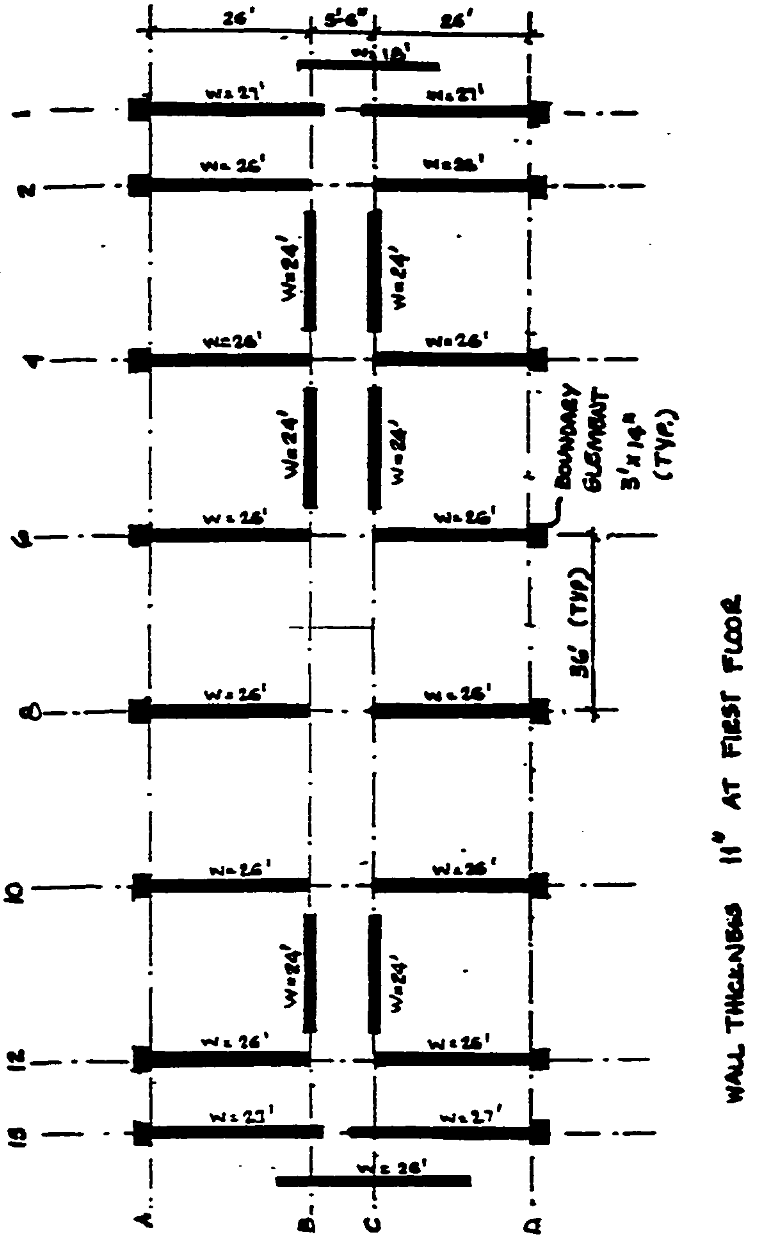
CSMIP Station No. 57355



A-1.10 Sketch of shear walls, San Jose Building

San Jose, 10-Story Residential Building

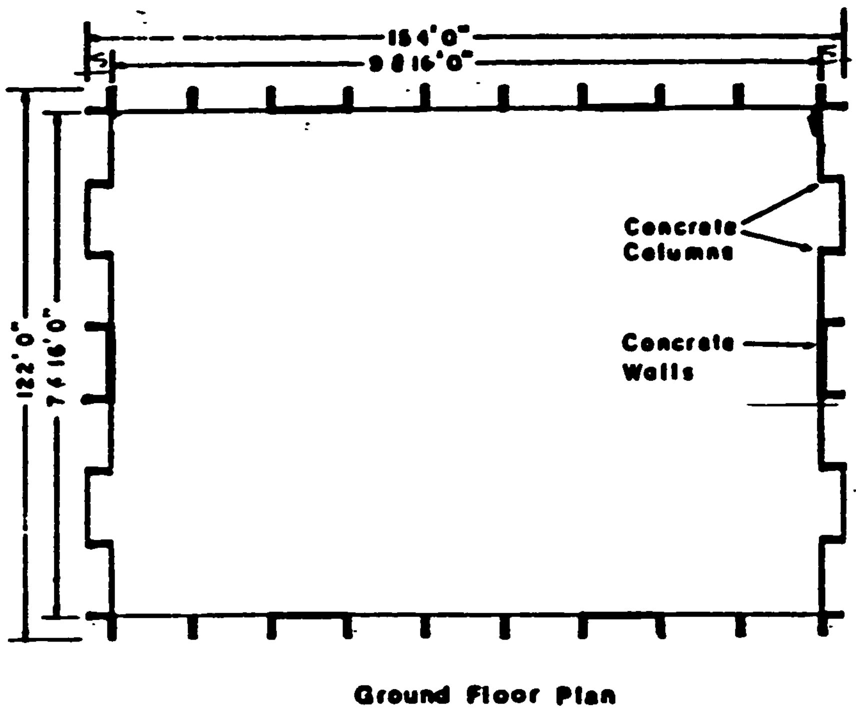
CSMIP Station No. 57356



A-1.11 Sketch of shear walls, San Jose Building

Saratoga, 1-Story School Gym

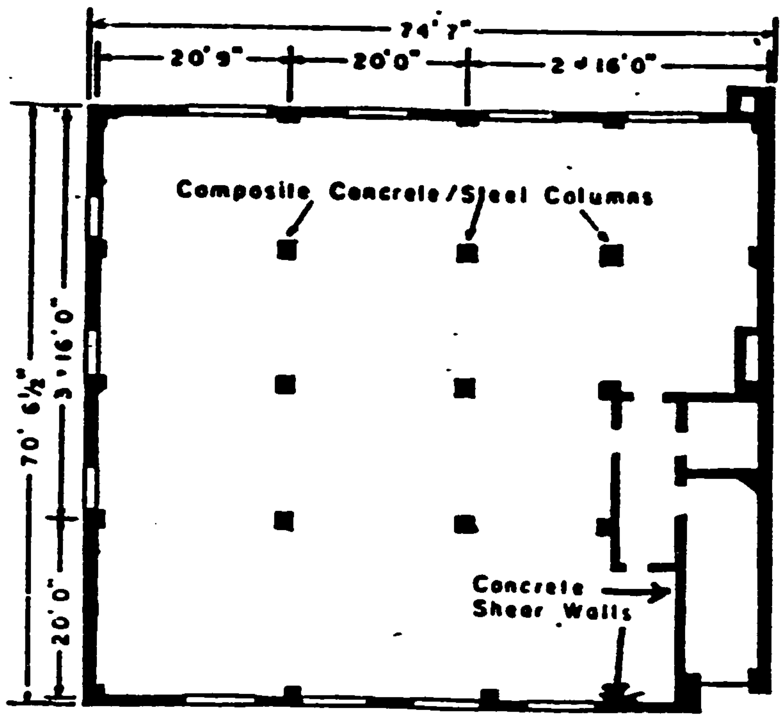
CSMIP Station No. 58235



A-1.12 Sketch of shear walls, Saratoga Building

Watsonville, 4-Story Commercial Building

CSMIP Station No. 47459



A-1.13 Sketch of shear walls, Watsonville Building