NATURAL PERIODS OF BRACED STEEL FRAMES DESIGNED TO EC8

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ABSTRACT

NATURAL PERIODS OF BRACED STEEL FRAMES DESIGNED TO EC8

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A two-phase study was undertaken to investigate the fundamental period of concentrically braced steel frames (CBFs) designed according to Eurocode 8. In the first phase, typical office buildings were studied by conducting two types of designs which are called as iterative and non-iterative. Non-iterative design is composed of obtaining final period by designing the structure with lower bound expression in Eurocode 8 while iterative design is similar to the non-iterative one but an updating of periods was considered in order to converge assumed and final periods. Different overstrength provisions are considered in the study. Lower bound expression in Eurocode 8 results in shorter periods which indicates that this expression can be safely utilized. The lower bound represented by Tremblay (2005) is also admissible except for some cases including shorter periods. In the second phase, a simple expression is derived for estimating the design base acceleration for braced frames proportioned according to Eurocode 8. This method requires inelastic top story drift values which were obtained from structures designed in the first phase using iterative method. These drifts were represented by simple expressions utilizing data fitting techniques. The method gives suitable first order estimate for the design base acceleration.

Keywords: Fundamental Period, Steel Frames, EC8 seismic provisions, Concentrically Brace

ÖZ

EC8 ŞARTNAMESİNE GÖRE DİZAYN EDİLEN ÇAPRAZLI ÇELİK ÇERÇEVELERİN DOĞAL PERİYOTLARI

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Eurocode 8'e göre dizayn edilmiş konsantrik çaprazlı çelik çerçevelerin temel periyodunun araştırılmasına ilişkin iki kısımlı bir çalışma ele alınmıştır. Birinci kısımda, tipik ofis binalar yinelemeli ve yinelemesiz olarak iki dizayn türüyle analiz edilmiştir. Yinelemesiz dizayn Eurocode 8'deki alt sınır denklemiyle dizayn edilen yapının nihai periyodunun elde edilmesinden oluşmaktadır, yinelemeli dizayn ise yinelemesize benzeyip farklı olarak nihai ve başlangıç periyotlarının birbirine yaklaşması için periyotların güncellenmesi düşünülmüştür. Farklı dayanım fazlası kuralları çalışmada göz önüne alınmıştır. Eurocode 8'de bulunan alt sınır denkleminin daima daha kısa periyotlar vermesi güvenle kullanabileceğini göstermektedir. Tremblay(2005) tarafından sunulan alt sınır denklemi de bazı daha kısa periyotlar içeren durumlar hariç uygun gelmektedir. İkinci kısımda, Eurocode 8'e göre boyutlandırılan çaprazlı çerçevelerin tasarım taban ivmesinin tahminini sağlayan basit bir denklem türetilmiştir. Bu yöntem birinci kısımdaki yinelemeli olarak dizayn edilen yapılardan elde edilen elastik olmayan üst kat ötelemelerine gereksinim duymaktadır. Bu ötelemeler veri uydurma yöntemi kullanılarak basit ifadelerle gösterilmiştir. Yöntem, tasarım taban ivmesi için uygun ilk tahminler vermiştir.

Anahtar Kelimeler: Temel Periyot, Çelik Çerçeveler, EC8 Sismik Kurallar, Konsantrik Çapraz

To My Family

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INTRODUCTION

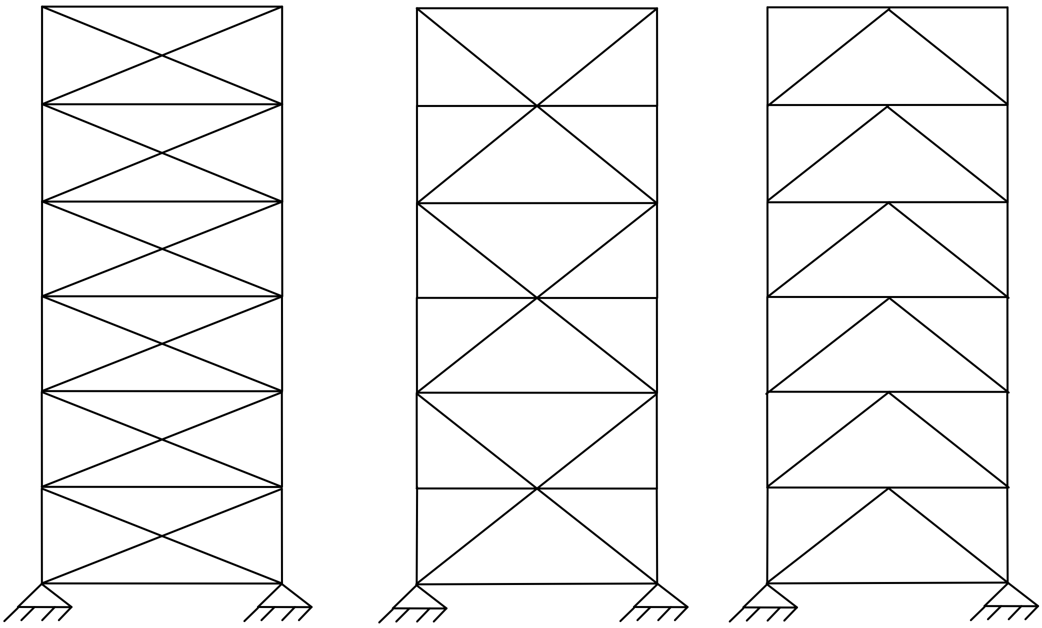
1.1 Description of Concentrically Braced Frames (CBFs)

Concentrically braced frames (CBFs) can be used as a lateral load resisting system in seismic regions. As shown in Figure 1.1, these systems resemble a vertical truss where the seismic forces are carried by axial loads produced on the members. The columns are for resisting overturning moments while the braces provide shear resistance. During an earthquake CBFs dissipate energy by yielding and buckling of the brace members. In order to ensure a satisfactory behavior the columns are designed to remain elastic during a seismic event.

Significant amount of structural damage was observed in steel moment resisting frames after Northridge and Kobe earthquakes. Until that time CBFs were not so common because these systems were considered to be non-ductile. After these earthquakes significant amount of research work has been undertaken to enhance the behavior steel lateral load resisting systems. Some of the research works have been tailored towards CBFs.

Design practice for CBFs is variable all over the world. AISC Seismic Provisions for Structural Steel Buildings (2005) classifies CBFs into two categories, namely Special Concentrically Braced Frames (SCBFs) and Ordinary Concentrically Braced Frames (OCBFs). Response modification factor and detailing requirements change depending on the type of system selected. Similarly Eurocode 8 classifies ductile lateral load resisting systems into two categories, namely Ductility Class Medium

(DCM) and Ductility Class High (DCH). According to Eurocode 8 there is no significant distinction for DCM and DCH CBFs and same behavior factor is assigned to both classes.



(a) X- Bracing (b) Split-X Bracing (c) Chevron Bracing

Figure 1.1: Typical Concentrically Braced Frames

The natural period of vibration for CBFs must be determined during the design stage in order to calculate the amount of earthquake forces. There are various methods used to calculate the natural period. The most accurate way of determining the period is to conduct an eigenvalue analysis. In addition, there are hand methods developed to predict the natural period. Furthermore, most specifications recommend lower bound expressions. These expressions are usually based on geometrical properties such as height and width and can be very useful in preliminary design stage before the member sizes are determined. The following sections outline the methods used to determine natural periods in general and the ones for CBFs in particular.

1.2 Code Formulas for Estimating Fundamental Periods of Structures

In ASCE 7-10, approximate fundamental period (T_a) (in sec) can be determined from the following formula:

$$T_a = C_t h_n^x \qquad (1.1)$$

Where h_n is the height of the structure, C_t and x can be obtained from Table 1.1

Table 1-1: Values of Approximate Period Parameters C_t and x for ASCE 7-10

Structural Type	C_t	x
Steel moment resisting frames	0.0724	0.8
Concrete moment resisting frames	0.0466	0.9
Steel eccentrically braced frames	0.0731	0.75
Steel buckling-restrained braced frames	0.0731	0.75
All other structural systems	0.0488	0.75

Periods obtained from a rational analysis can be used according to ASCE7-10. However, the period obtained from a rational method cannot be larger than a factor multiplied by the approximate period obtained using Equation 1.1. These factors depend on the level of seismicity. It is worthwhile to note that periods from a rational analysis can be directly used without upper bounds in checking drift requirements.

The Turkish Earthquake Code (TEC07) does not present any empirical formula but recommends a formulation based on Rayleigh's method. As a result, this method requires the mass and displacements of the structure under a fictitious load. Therefore, designer must perform a first trial design to obtain these parameters. In TEC07, fundamental period of any type of building might be calculated as follows:

$$T_1 = 2\pi \left(\frac{\sum_{i=1}^{N} m_i . d_{fi}{}^2}{\sum_{i=1}^{N} F_{fi} . d_{fi}} \right)^{1/2} \tag{1.2}$$

Where $m_i = (g_i + nq_i)/g$

gi: dead load

qi: live load

n: is the category of use for live load reduction

d_{fi}: deformations calculated under fictitious loads in the i_{th} storey

F_{fi}: the fictitious load acting on i_{th} storey in the first natural vibration period calculations.

In Eurocode 8 any rational method can be used to calculate the natural period of vibration. Alternatively some empirical formulas are also presented. For CBFs Eurocode 8 has a similar treatment when compared with ASCE7-10. The same empirical formulation with slight modifications is recommended. It is stated that for buildings with heights up to 40 m the value of T_1 (in sec) can be approximated by the following expression.

$$T_1 = C_t . H^{3/4} \tag{1.3}$$

Where

C_t: is 0.085 for moment resistant space steel frames, 0.075 for eccentrically braced steel frames and 0.050 for all other structures;

H: is the height of the building, in m, from the foundation of or from the top of a rigid basement.

Alternatively, the estimation of T_1 (in sec) may be made by using the following expression:

$$T_1 = 2.\sqrt{d} \tag{1.4}$$

Where

d: is the lateral displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction.

2005 National Building Code of Canada offers a more simplified expression to calculate natural periods of CBFs. The expression given in Equation 1.5 was developed by Tremblay (2005) and details of his work will be presented in the following sections.

$$T_a = 0.025 h_n \tag{1.5}$$

Where h_n (m): building height

T_a: fundamental period of the structure

1.3 Past Research on Determination of Structural Periods

Research can be divided into two categories. Some researchers developed simplified expressions while the others developed hand techniques.

1.3.1 Simplified Methods in Calculating Periods

G.W.Housner and A.G.Brady (1963)

This is one of the first studies on determining structural periods. Simplified equations are derived for fundamental periods of idealized buildings and these are compared with the measured ones.

For shear-wall buildings, none of the simple empirical equations give precise estimates unless wall stiffness is involved in period calculations. If design of a building is affected by calculated period of vibration, it is recommended that the period is computed by Rayleigh method or estimated by a reference to the measured period of a similar building. Calculated periods of steel frames are described with the formula T=$1.08\sqrt{N}-0.86$ where N is the number of stories. The measured periods of some modern steel structures can be described as follows T=$0.5\sqrt{N}-0.4$. It is concluded that period of vibration of a structure is the most informative parameter about the internal structure of building.

It is observed that when using California building codes, period estimation of buildings with shear walls is less accurate than other structural types. The paper also shows that a precise estimation of period of a structure is not possible with simple empirical expressions.

Goel and Chopra (1997)

The purpose of this paper is to develop code formulas which calculate periods of structures by means of recorded motions statistics. This study includes RC and steel moment-resisting frames. A regression analysis of measured data was made to improve code formulas for estimating fundamental periods of structures.

Building database contains 106 California buildings including 21 buildings' with (Peak Ground Acceleration) PGA>0.15g. It is observed that calculated code periods are shorter than the measured periods from the recorded motions. For buildings up to 36 m, code formulas give approximately lower-bound values of measured period data; on the other hand they result in 20-30% shorter compared to measured data for buildings taller than 36 m. For many buildings, measured period values are bigger than 1.4T where T is period value obtained from empirical formulas. This means that code limits on the rational analysis period results are too restrictive. It is stated that the database must be expanded with the new earthquake data.

R. Tremblay (2004)

The aim of this study is to propose a simple expression for the fundamental period of CBFs by performing an analytical study. Besides, available test and field data of building periods were compared to ones obtained with analytical predictions. Building and design parameters were examined through a closed form solution and an extensive parametric study.

From the simplified closed-form model, it is concluded that large variations on structure periods are related with differences in seismic zones. Therefore, it is not possible to capture these differences with the simple expressions that could calculate initial periods in building codes. But, it is feasible to use such crude code expressions provided that they give lower-bound results.

This work also includes a parametric study on braced steel frames. 7524 buildings have been analyzed in this study. Braced frame tributary areas were 250, 500 and 1000 m^2. Number of stories varied from 1 to 25. Story height was taken as 4 m.

The fundamental period of concentrically braced frames was found to vary with the frame geometry and the magnitude of the design seismic loads, the latter being a function of the seismic hazard level and soil conditions at the site as well as of the period, the force modification factor, and the importance factor that are used in design. A regression analysis is made by using the scatter of calculated periods lying between $T=0.025h_n$ and $T=0.09h_n$ where h_n is total building height. The best fit expression is $T=0.056h_n^{0.9}$. Considering a linear variation, best fit becomes $T=0.04h_n$, best fit minus one sigma is $T= 0.030h_n$. It is found that selection of which expression is used has small effects on the design of the structure. Besides, when the importance factor increases, fundamental periods decrease as expected.

It is confirmed that most effective factor on the period is the building height. Braced frame width also affects the period but not as much as other parameters. It is discovered that CBFs period values cannot be estimated with only building height and frame width which take place of the simple expressions. Building periods mostly increase with weight of the structures but this has small effects on the periods. Because, periods change approximately 5% considering different structure weights.

Study shows that period values of CBFs increase with the building height linearly. As a lower-bound estimation, $T_a=0.025h_n$ is recommended according to this study.

Rui Pinho and Helen Crowley (2009)

This paper evaluates estimating period of vibration of reinforced concrete MRFs in various codes around the world while performing linear static and dynamic analyses. Effect of period on the structural design is discussed shortly and some improvement of period estimating in Eurocode 8 is made. It is observed that there is a big difference between stiffnesses of pre-1980 buildings and post-1980 buildings because of changes of design philosophy; new buildings were found to be stiffer. Therefore, Eurocode 8 period equation matches well with the measured periods of new buildings erected in Europe.

Eurocode 8 allows lateral force method to be carried out for buildings whose response is not broadly affected contribution of higher mode vibration. If higher mode contribution becomes effective for the structure, a modal response spectrum analysis should be applied to be more realistic. Recent studies have shown that these two types of methods differ with calculated design base shear forces for a given building. This difference mainly rises from calculated periods which are period obtained by period-height equation for lateral force method and period of vibration from eigenvalue analysis. So, many codes realize that simplified period-height equation looks more realistic unless higher mode effects become necessary. Some codes suggest that if the modal base shear is less than 85% of the lateral force method base shear, modal forces should be multiplied with $0.85V/V_t$ where V is base shear of lateral force method and V_t is modal base shear. This coefficient will be a safeguard to avoid from low forces of analytical models with unrealistically high periods of vibration.

Oh-Sung Kwon and Eung Soo Kim (2010)

This paper is an evaluation of seismic code period formulas applied to 800 actual buildings. The ASCE 7-05 code is investigated and an evaluation is performed for RC and steel moment resisting frames, braced frames, shear wall buildings and other structural types. Database contains 34 concentrically braced frames (CBFs) and 125 steel moment resisting frames and other structural types.

From comparison of measured periods and code equation periods, equation of steel MRFs estimates well lower bound of the measured periods for all building heights. This difference is relatively high for low-to-medium rise buildings. Moreover, the periods of essential buildings with bigger importance factors result in 40% shorter periods than the non-essential building periods. The code formula for braced frames estimates lower bound periods for low-to-medium rise buildings.

Based on the limited available data for CBFs, code formula tends to underestimate lower bound of the periods of structures taller than 61 m. Code formula provides good estimates of periods for low-to-medium rise buildings.

1.3.2 Hand Methods in Calculating Periods

Bryan Stafford Smith and Elizabeth Crowe (1985)

A hand method that can be used in estimating periods of a structure was developed. Structures which are analyzed with this method must be regular in plan and also along the height. Moreover, structure must be loaded symmetrically not to impose a torsion effect. The method is based on a technique that regards coupled walls, rigid frames, braced frames and wall-frames as a shear-flexure structure so that their static deflection can be determined with coupled wall theory. It is useful to decouple static deflection into two parts as flexural component and shear plus flexure, so that dynamic behavior of the structure can be captured by decoupled eigenvalue approach.

Results of this study can be summarized for braced frames as follows:

First natural period can be obtained by this method with a 2.9% error. For all structure types, second mode of vibration period can be estimated with a 15% error.

K.A.Zalka (2001)

This is a simple hand method that performs a three-dimensional frequency analysis of structures such as coupled shear walls, shear walls and cores. Lateral vibration was defined by three deformation types such as full height local bending, full height global bending and shear deformation of the frameworks. The aim of the study was to develop a closed from solution to estimate lateral frequencies by using their stiffnesses.

Like other studies, it is assumed that structures are regular along the height. Accuracy of the method was checked with the finite element solution results. 4, 10, 16, 22, 28, 34, 40, 60, 80 storey buildings with eight different frameworks are used in the comparison. Storey height was 3m and the bays of the frameworks were 6 m. For the 144 cases, the average error between the hand method and finite element solution was around 2% and with a maximum error of 7%.

It has been shown that this closed-from solution can be used for the calculation of the natural frequencies of multi-storey buildings.

C.Chrysanthakopoulus, N.Bazeos and D.E.Beskos (2005)

This paper is an extension of the method developed by Bryan and Crowe (1985) to CBFs and MRFs. This method considers plane steel frames as a flexural-shear cantilever beam as discussed before. It is valid for both braced and unbraced frames. It is possible to obtain the first three natural period of the structure with this method. It employs many parametric studies which includes 110 braced and unbraced frames analyzed with finite element method to establish a formula that reflects the character of equivalent cantilever beam. Stafford Smith and Crowe’s study is extended by some correction factors by using parametric studies. Maximum 15-story frame is considered in this work. This study considers non-uniform member properties along the height.

This method was found to be simple and conservative enough for performing a hand calculation in determining first three natural periods. Maximum error is calculated as

15% for the first and second natural periods, and third period can be estimated within 20% accuracy. The method is limited for only one braced bay, it requires more computational work to establish a formula for more than one braced bay. Generally, story height and bay width affects the accuracy of the estimation of the period slightly.

1.4 Scope of the Thesis

As explained in the earlier sections of the thesis, there are various methods for calculating the natural periods of braced frames. Eurocode 8 has very special rules for proportioning the CBFs and the resulting member sizes can be quite different compared to designs conducted in other parts of the world.

The study aims to evaluate the fundamental periods of CBFs designed according to Eurocode 8. A two phase research study has been undertaken. In the first phase, several CBFs were designed according to Eurocode 8 and the accuracy of the empirical formulas is evaluated. In the second phase, a more accurate empirical equation was developed based on the database that was formed in the first phase. This new empirical equation takes into account various factors such as the level of seismicity, soil conditions, gross dimension, mass properties and etc.

The detailed rules for proportioning CBFs using Eurocode 8 are presented in Chapter 2. The evaluation of empirical formulas using a CBF database is given in Chapter 3. Chapter 4 outlines the development of a new empirical relationship and presents its accuracy on the CBFs designed in Chapter 3. Finally, Chapter 5 presents the conclusions derived from this two-phase study.

CHAPTER 2

DESIGN OF CONCENTRICALLY BRACED STEEL FRAMES ACCORDING TO EURONORMS

2.1 Calculation of Lateral Loads According to EC8

2.1.1 EC8 Design Spectrum and Identification of Ground Types

Ground types A, B, C, D, and E may be used to account for the influence of local ground conditions on the seismic action. These ground types are classified in Eurocode 8.

Soil type A represents rock or other rock-like material formation, including at 5 m most of weaker material at the surface. Soil type B accounts for deposits of very dense sand, gravel, or very stiff clay. Ground type C is defined as deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. Type D represents deposits of loose-to-medium cohesionless soil. Type E is a soil profile consisting of a surface alluvium layer with average shear wave velocity values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material according to Eurocode 8.

If there is not sufficient research on the geology of the seismic zone, Eurocode 8 suggests two types of spectra, Type 1 is valid for earthquakes that have surface-wave magnitude (M_s) greater than 5.5, while Type 2 represents earthquakes whose M_s values are not greater than 5.5. In this study, Type 1 spectrum is adopted. The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type for Type 1 spectra. The values recommended in Eurocode 8 for T_B, T_C, T_D, S are given in Table 2.1.

Table 2-1: Values of Parameters about Soil Conditions

Ground Type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.2	0.6	2.0
D	1.35	0.2	0.8	2.0
E	1.4	0.15	0.5	2.0

Where;

S soil factor;

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum

2.1.2 Design Spectrum

For the horizontal components of the seismic action the design spectral acceleration, $S_d(T)$, is defined by the following expressions in Eurocode 8;

$$0 \le T \le T_B: \quad S_d(T) = a_g.S.\left[\frac{2}{3} + \frac{T}{T_B}.\left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{2.1}$$

$$T_B \le T \le T_C: \quad S_d(T) = a_g.S.\frac{2.5}{q} \tag{2.2}$$

$$T_C \le T \le T_D: \quad S_d(T)\begin{cases} = a_g.S.\dfrac{2.5}{q}.\left[\dfrac{T_C}{T}\right] \\ \ge \beta.a_g \end{cases} \tag{2.3}$$

$$T_D \leq T: \quad S_d(T)\begin{cases} = a_g.S.\dfrac{2.5}{q}.\left[\dfrac{T_C.T_D}{T^2}\right] \\ \geq \beta.a_g \end{cases} \tag{2.4}$$

Where;

a_g is the design ground acceleration on type A ground

T is the vibration period of a linear single-degree-of-freedom system

β is the lower bound factor for the horizontal design spectrum, its recommended value is 0.2.

q is the behavior factor

There is one single parameter to describe seismic hazard for most applications in Eurocode 8, i.e. the value of reference peak ground acceleration (a_{gR}) on type ground A. The value of reference peak ground acceleration is chosen by national authorities for each seismic zone. The design ground acceleration is calculated as follows:

$$a_g = \gamma_I \times a_{gR} \tag{2.5}$$

Where;

γ_I is the importance factor

In Eurocode 8 Table 4.3, importance class II is defined as ordinary buildings, not belonging other categories. γ_I equals to unity is considered herein which is for importance class II.

q is the behavior factor which represents ductility capacity of a structure. In Eurocode 8 Table 6.2, upper limit of the behavior factor for concentrically braced frames corresponds to 4. This value is valid for both Ductility Class Medium (DCM) and Ductility Class High (DCH). The design spectra for different soil types are given in Figure 2.1.

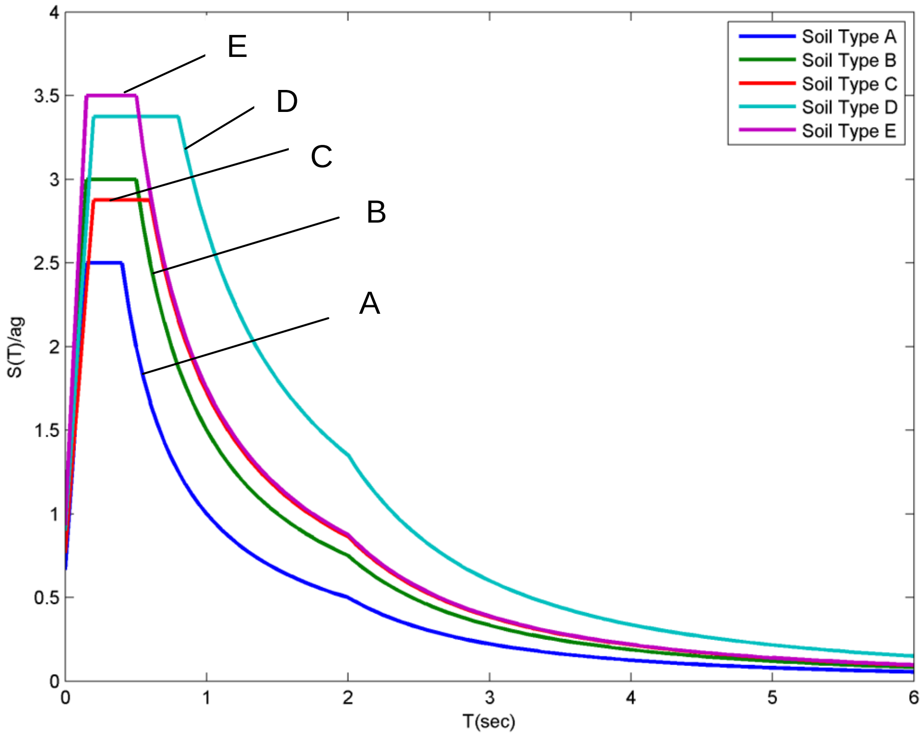


Figure 2.1: Design Spectra for q=1

2.2 Calculation of Base Shear

Several methods can be used to calculate the seismic actions according to Eurocode 8. These are equivalent lateral force method, response spectrum analysis and time history analysis. Equivalent lateral static force method is used to calculate seismic forces in this thesis. This type of analysis can be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

In order to use this method in the analyses, structures must satisfy the following requirements;

a) They have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values.

$$T_1 \leq \begin{cases} 4T_c \\ 2.0 \sec \end{cases} \tag{2.6}$$

Where;

T_1 is the fundamental period of the building in the horizontal direction of interest. As described in Chapter 1, for buildings with heights of up to 40 m the value of T_1 (in sec) may be approximated by Equation 1.3. Although there is a height limit on Equation 1.3, it is used throughout this thesis without a limit.

b) They meet the criteria for regularity in elevation

Base shear (F_b) acting on a structure is calculated with the following formula as it is stated in Eurocode 8 provisions.

$$F_b = S_d(T_1).m.\lambda \tag{2.7}$$

Where;

m is the total mass of the building, λ is the correction factor which is a value that is equal to: $\lambda=0.85$ if $T_1 \leq 2.T_c$ and the building has more than two stories, or $\lambda=1$ otherwise. Mass calculations are performed with the following formula in Eurocode 8;

$$\text{m}=\sum G_{k,j} \text{"+"} \sum \psi_{E,i}.Q_{k,i} \tag{2.8}$$

Where;

$\sum G_{k,j}$ is the permanent action like self-weight of the structure, $Q_{k,i}$ is the variable action like normal use by persons, furniture and moveable objects, $\psi_{E,i}$ is the combination coefficient for variable action which is determined as follows:

$$\psi_{E,i} = \psi_{2,i}.\varphi \tag{2.9}$$

φ is a factor obtained from Table 4.2 of Eurocode 8. For independently occupied stories and category B (office buildings) corresponds to 0.3 for φ. $\psi_{2,I}$ is the combination coefficient for the quasi-permanent value of a variable action I, which is obtained from EN1990, Table A1.1 for category of use B. The resulting $\psi_{E,I}$ is 0.15 for the cases studied in this thesis.

After calculating the base shear, lateral forces should be distributed along the height of the structure. The following formula is used which produces linearly increasing lateral loads for cases with equal mass in all stories.

$$F_i = F_b \frac{z_i . m_i}{\sum z_j . m_j} \tag{2.10}$$

Where;

F_i is the horizontal force acting on story I;

m_i, m_j are the story masses computed in accordance with Eqn. 2.8.

z_i, z_j are the heights of the masses m_i m_j above the level of application of the seismic action (foundation or top of a rigid basement).

The horizontal forces F_i determined in accordance with this clause shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane.

2.3 Torsional Effects

In order to account for uncertainties in the location of masses and spatial variation of the seismic motion, center of mass can be considered as being displaced from its calculated location in each direction by an accidental eccentricity as follows:

$$e_{ai} = \pm 0.05 L_i \tag{2.11}$$

Where;

e_{ai} is the accidental eccentricity

L_i is the floor-dimension perpendicular to the direction of the seismic action

According to Eurocode 8, if the masses and lateral stiffness are symmetrically distributed in plan and unless torsional effects are taken into consideration by a more exact solution, the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of distribution of the horizontal seismic forces (F_i) by a factor δ. This factor amplifies the base shear to account for torsional effects and can be calculated as follows:

$$\delta = 1 + 0.6\frac{x}{L_e} \tag{2.12}$$

Where;

x is the distance of the element under consideration from the center of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered;

L_e is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

For the cases studied herein the x/L_e term is considered as 0.5 for the braced frames that are located at the perimeter of the building. Therefore, a torsional amplification factor of 1.3 is considered for all cases.

2.4 Design of Steel Members According to EC3

2.4.1 Design of Members for Tension According to EC3

Only gross section yielding was considered in this study as a tension member limit state, according to EC3 the following condition must be satisfied:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{2.13}$$

Where;

N_{Ed} is design value of the axial tension force

$N_{t,Rd}$ is design value of the resistance to tension forces

The design value of resistance can be determined as follows:

$$N_{t,Rd} = \frac{A.f_y}{\gamma_{M0}} \tag{2.14}$$

Where;

A is the gross cross-sectional area

f_y is the yield strength

γ_{M0} partial factor for resistance of all cross-sections classes and is taken as unity

2.4.2 Design of Members for Compression According to EC3

The following condition must be satisfied to design compression members against instability failure:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \tag{2.15}$$

Where;

N_{Ed} is the design value of compression force

$N_{b,Rd}$ is the buckling resistance capacity

The design resistance considering instability effects is calculated as follows:

$$N_{b,Rd} = \frac{\chi . A . f_y}{\gamma_{M1}} \tag{2.16}$$

Where;

χ is the reduction factor for the relevant buckling mode

γ_{M1} is a partial factor accounts for resistance of members for buckling

The recommended value of γ_{M1} is equal to 1 according to EC3. Reduction factor and non-dimensional slenderness (λ) for the relevant buckling mode can be calculated as follows:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} \text{ but } \chi \leq 1.0 \tag{2.17}$$

$$\Phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] \tag{2.18}$$

$$\lambda = \sqrt{\frac{A . f_y}{N_{cr}}} \tag{2.19}$$

Where;

N_{cr} is the elastic critical force for relevant buckling mode

α is the imperfection factor which varies according to cross section geometries and types

α can be obtained from Table 6.1 and Table 6.2 in EC3 for the relevant buckling curve, cross-sections and grade of steel. For the cross-section types considered in this study, values of α varied between 0.13, 0.21, 0.34, 0.49, 0.76.

Elastic critical force can be obtained from the following formula:

$$N_{cr} = \frac{\pi^2 EI}{(KL)^2} \tag{2.20}$$

Where;

E is the elastic modulus of steel

I is the moment of inertia of the member considered

L is member length

K is the effective length factor

Effective length factor was taken as “1” since all members were considered to be pin connected.

2.5 Design Rules for Concentrically Braced Frames According to EC8

Concentrically braced frames must be designed in such a way that dissipative zones take place in diagonals in tension before columns and beams yield and buckle.

Majority of the lateral loads that act on the frame are carried by columns and braces in the braced bay. Because all connections are pinned, the entire frame can be considered as if it has one single braced bay. Dissipative zones should be located in the tensile diagonals, therefore it is assumed that the compression diagonals already buckle according to Eurocode 8. In other words, horizontal forces can be resisted only by the tension diagonals without considering compression diagonals. (Fig. 2.2)

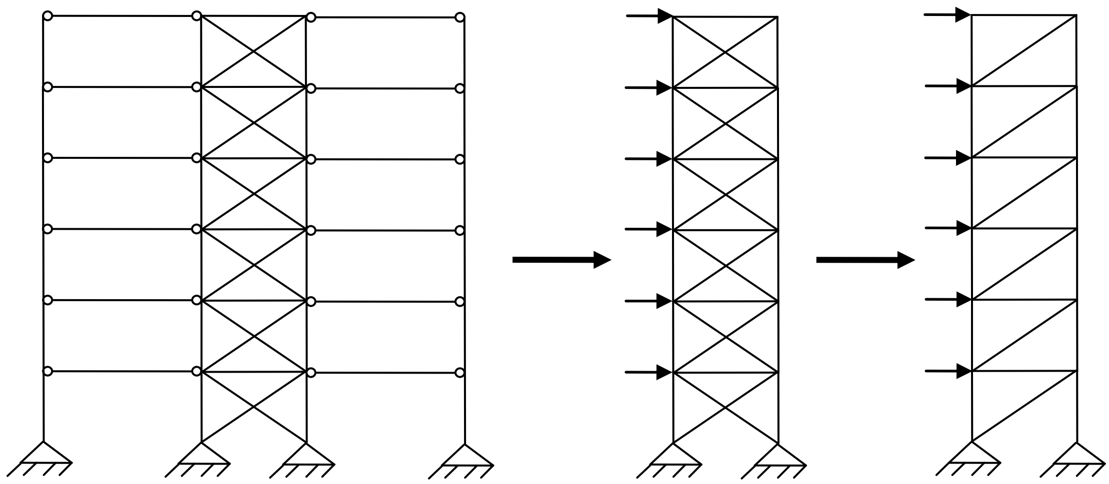


Figure 2.2: Assumptions for Modeling the Frame

2.6 Design Rules for Diagonal Members

For the frames with X-bracing systems, the non-dimensional slenderness for all diagonals is limited by the following constraint.

$$1.3 < \lambda \leq 2.0 \tag{2.21}$$

The lower bound of 1.3 is enforced for preventing overloading of columns in the pre-buckling stage (when considering both compression and tension diagonals). The upper bound is placed to prevent any shock effects.

The yield resistance must satisfy Equation 2.13. Unlike many other design codes, EC8 enforces uniform yielding along the height of the structure for a more homogenous energy dissipation behavior. Therefore overstrength (Ω) of braces should be close to each other. According to EC8 the maximum overstrength (Ω_{max}) does not differ from the minimum overstrength (Ω_{min}) by more than 25 %.

$$\Omega_{i} = \frac{N_{pl,Rd,i}}{N_{Ed,i}} \tag{2.22}$$

Where;

$N_{pl,Rd,I}$ is the design resistance of diagonal i

$N_{Ed,I}$ is the axial force in the same diagonal according to design seismic situation

2.7 Design Rules for Columns

Columns shall satisfy the following formula for columns with axial forces in concentrically braced frames.

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.N_{Ed,E} \quad (2.23)$$

Where;

$N_{pl,Rd}(M_{Ed})$ is design buckling resistance value of the column

$N_{Ed,G}$ is the axial force of the column due to non-seismic effects calculated with combinations of actions for the seismic design situation

$N_{Ed,E}$ is the axial force in the column because of the design seismic situation

γ_{ov} is the material overstrength which is equal to 1.1 for steel grade S355

Ω is the minimum value of overstrength obtained according to Eqn. 2.22

2.8 Calculation of Brace & Column Forces

In EN1990, combinations of actions for the seismic design situations are described as follows:

$$\sum_{j\geq 1} G_{k,j} \text{"+"} A_{Ed} \text{"+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \quad (2.24)$$

Where;

A_{Ed} is the design value of the seismic action

$$A_{Ed} = \gamma_I . A_{Ek} \quad (2.25)$$

Where;

A_{Ek} is the characteristic value of the seismic action

$\psi_{2,I}$ equals to 0.3 because category of use is B (office areas) in EN1990. Eqn. 2.24 turns into following formula:

$$G_{k,j} + A_{Ed} + 0.3Q \tag{2.26}$$

2.9 Lateral Drift Check

After the design is performed, displacements are calculated and compared against the drift limits. For buildings having non-structural elements fixed in a way so as not to interfere with structural deformations the following must be satisfied:

$$d_r \nu \leq 0.010h \tag{2.27}$$

Where;

d_r is the design interstory drift, considered as difference of the average lateral displacements (d_s) at the top and bottom of the story

h is the story height

ν is a reduction factor which takes into consideration the lower return period of the seismic action

d_r are defined as follows by Eurocode 8:

$$d_r = q \times d_s \tag{2.28}$$

Where;

q is the behavior factor

d_s is the displacement of the structure designed by linear analysis based on the design spectrum mentioned in 2.1.2.

ν is related with the importance factor. Its recommended values are 0.5 for importance classes I and II, and 0.4 for importance classes III and IV. Equation 2.27 translates into a maximum of 2% lateral drift ratio (d_r/h) for importance class II.

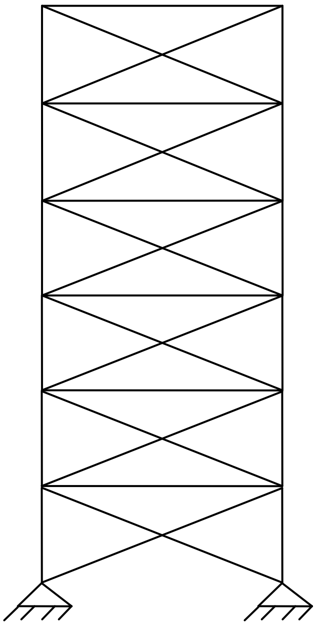
CHAPTER 3

EVALUATION OF EMPIRICAL CODE FORMULAS

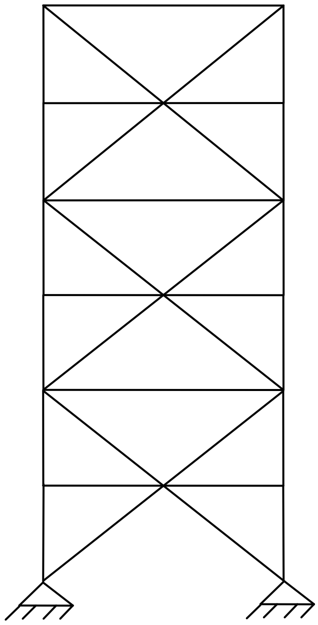
The accuracy of the empirical lower bound expressions given in well-known specifications are evaluated in this chapter. For this purpose a MATLAB program was developed. This program is capable of conducting designs for different braced frame configurations. After the design is complete the program determines the fundamental period of vibration using the Rayleigh's method. In general, the rules presented in Eurocode 8 and Eurocode 3 were strictly followed. For each of the geometry considered two separate designs were conducted. As explained before, a trial period needs to be selected to come up with an initial design. All designs start by assuming a fundamental period defined by Equation 1.3. The difference between the two design methods arises from the way in which the fundamental period is updated. In the first method the periods were not updated. In other words the lateral forces are calculated based on the period obtained from Equation 1.3. On the other hand, the second method is an iterative design and uses updating of periods. Because Eurocode 8 allows for any rational method to be used, the Rayleigh's method can be adopted to calculate the periods. The iterative method starts with an initial period taken equal to the period given by Equation 1.3. A design is completed based on this initial period. Later, the actual period of the structure is calculated by Rayleigh's method and the design is updated by making use of this new period value. Design process is continued until the initial and the final period converges. The second method is more realistic because the lateral forces are calculated based on the fundamental period obtained from a rational analysis. In general, iterative type of designs results in lighter and cost efficient systems. The structural layouts are presented next followed by the analysis results.

3.1 Geometrical Properties and Layouts of the CBFs Considered

Two types of bracing systems were considered in this study. The first system is concentrically X-bracing (Fig. 3.1a), and the other one is Split X-bracing (Fig. 3.1b).



(a) X-bracing system



(b) Split-X bracing system

Figure 3.1: Bracing Systems (a) X-bracing system, (b) Split-X bracing system

Dead load is taken to be equal to 4.4 kN/m^2 or 9 kN/m^2. All buildings were considered to be used as office buildings. This usage category corresponds to type B according to Table 6.1 in EN1991. The recommended value of imposed loads for Category B corresponds to 2 - 3 kN/m^2 in Table 6.2 of EN1991. Therefore, live load intensity is taken as 2 kN/m^2. Plan views of the buildings considered in this study are given in Figs 3.2 through 3.7. Six types of floor plan systems are considered. They vary with respect to their number of braced bays and braced frame tributary area. These plans include two braced bays or four braced bays.

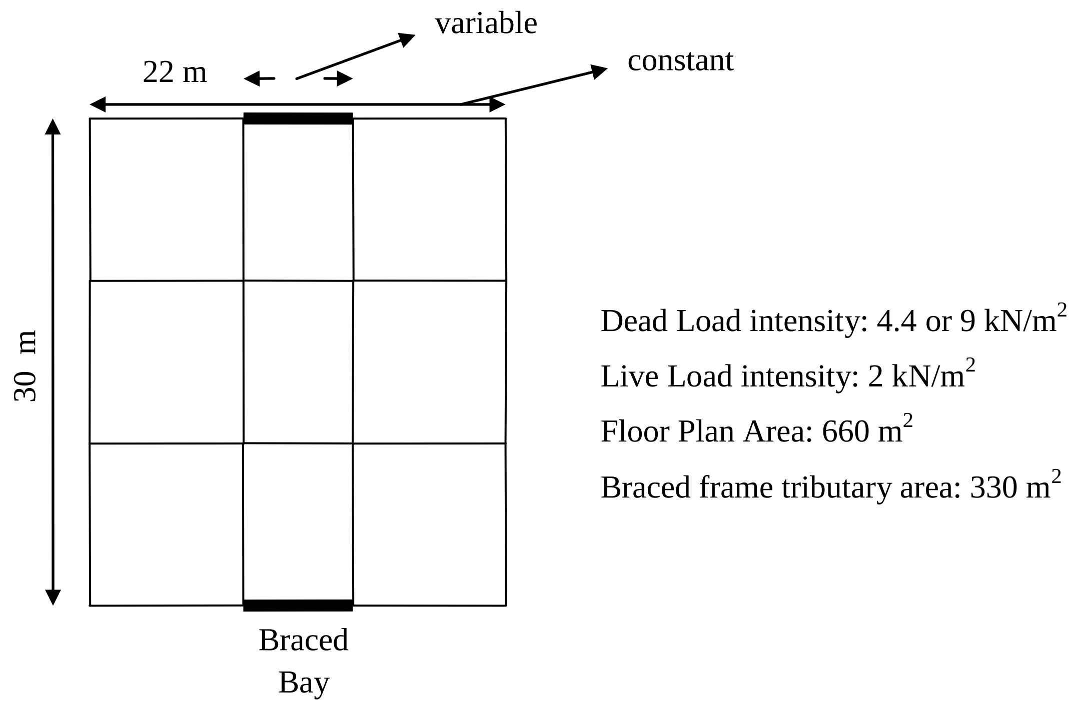


Figure 3.2: Plan View of the Building with Two Braced Bays with BFTA of 330 m^2

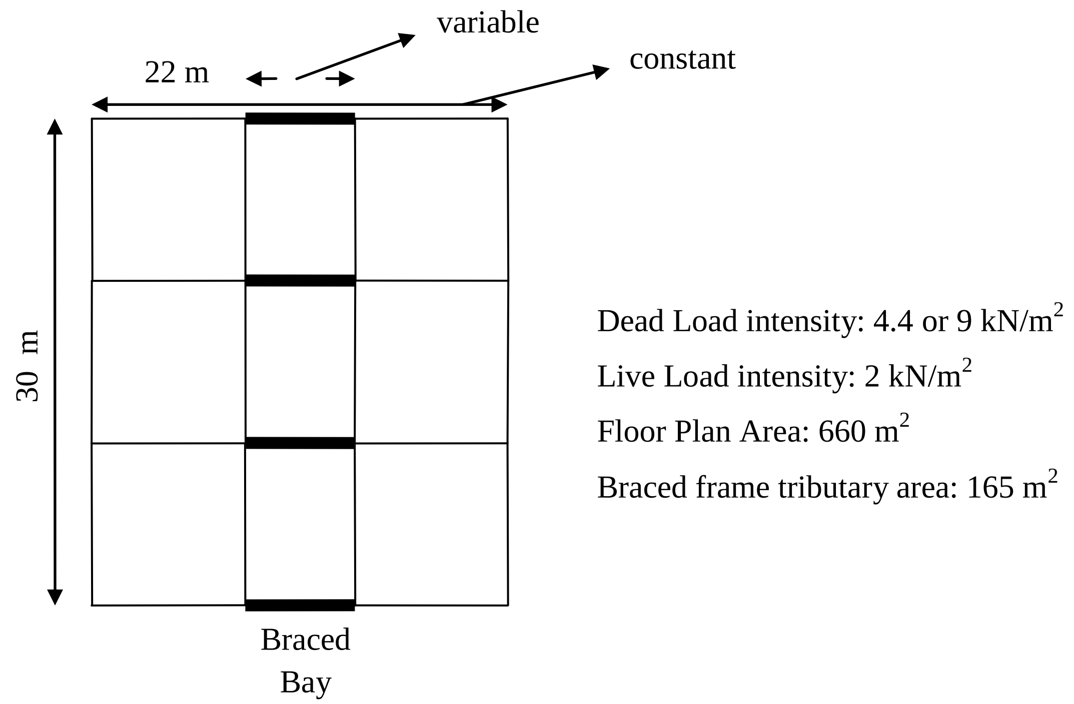


Figure 3.3: Plan View of the Building with Four Braced Bays with BFTA of 165 m^2

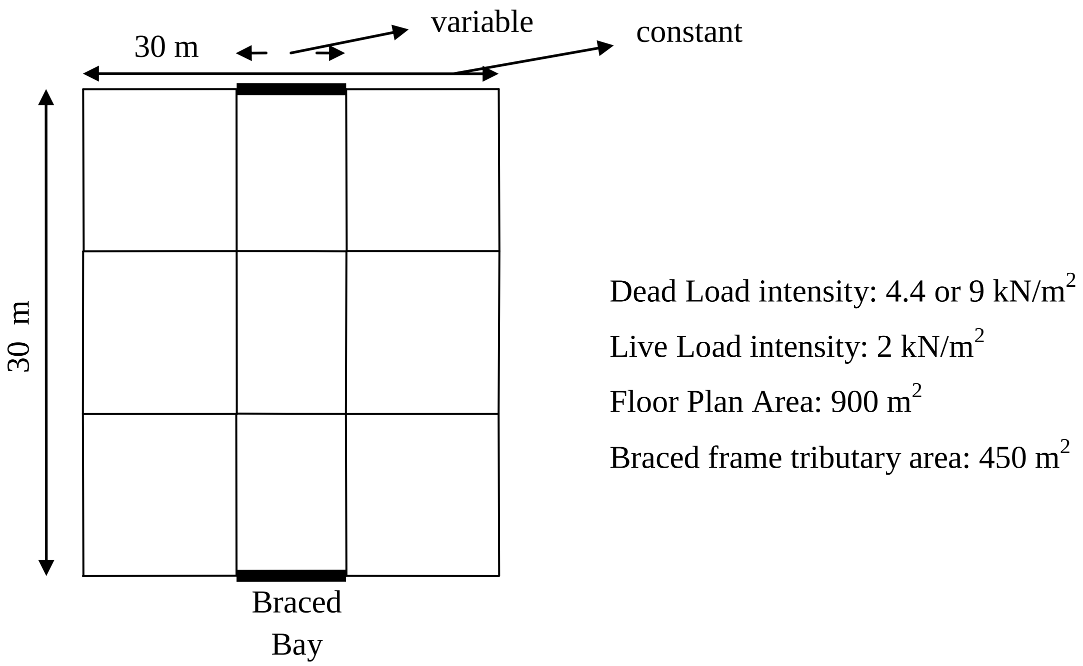


Figure 3.4: Plan View of the Building with Two Braced Bays with BFTA of 450 m^2

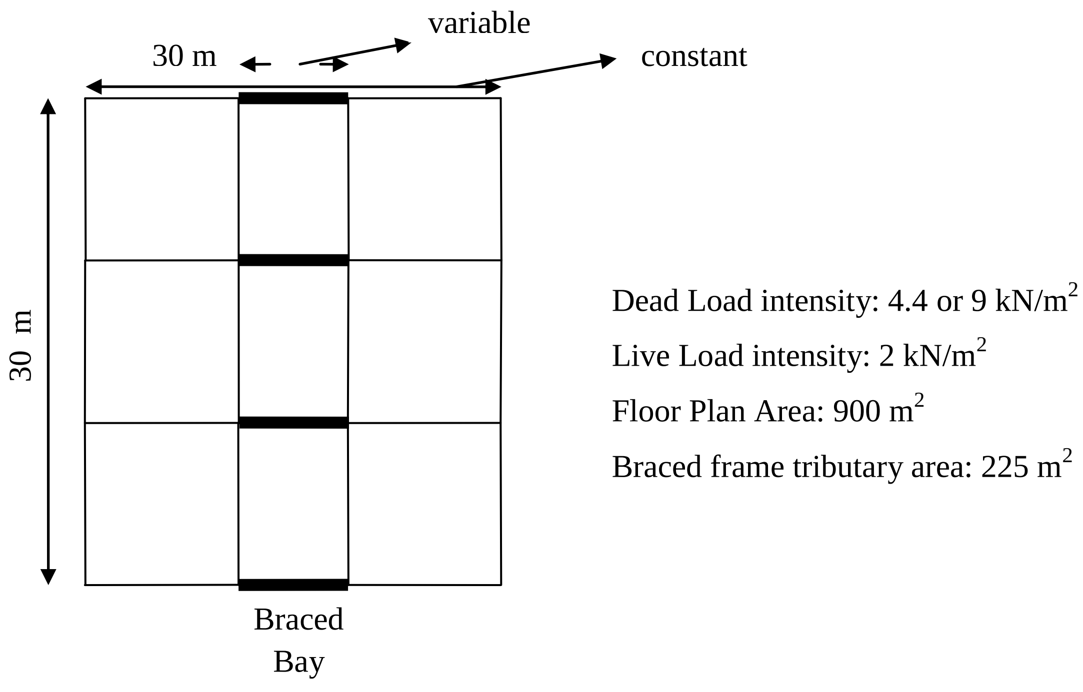
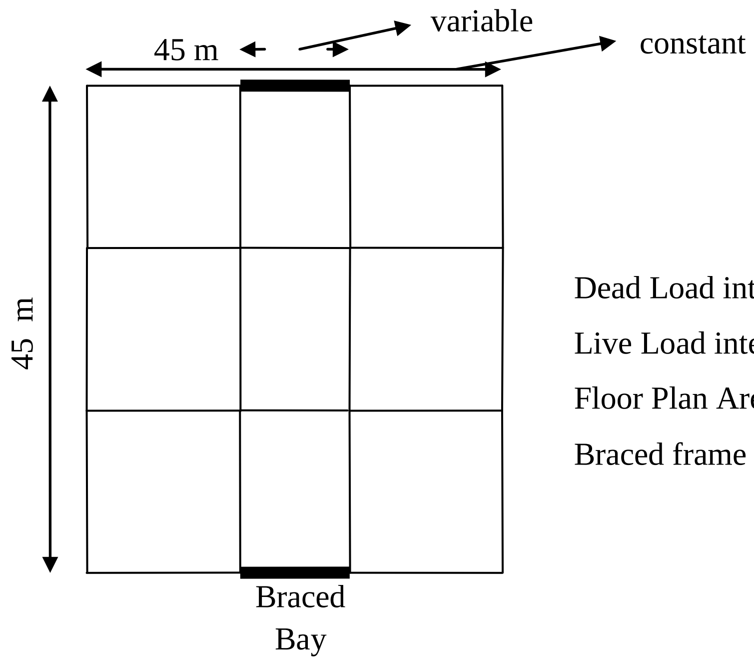


Figure 3.5: Plan View of the Building with Four Braced Bays with BFTA of 225 m^2



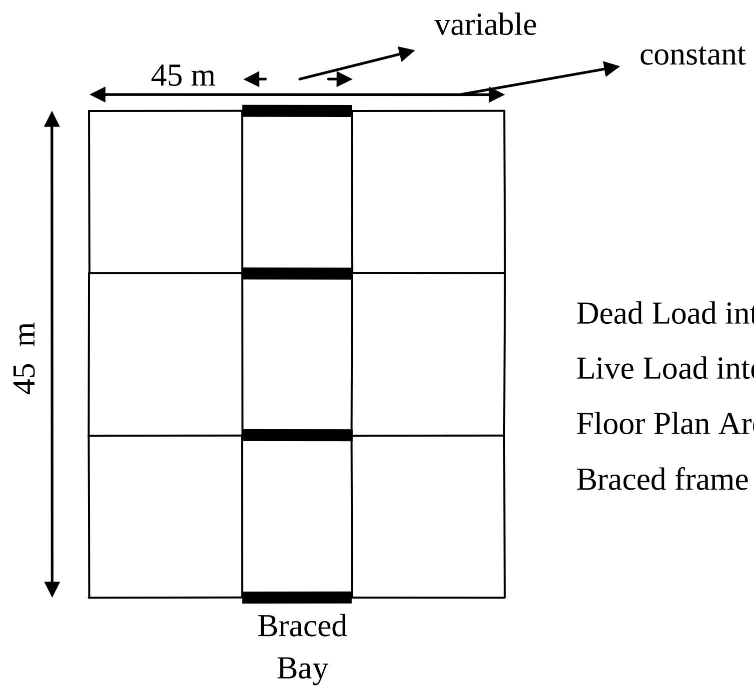
Dead Load intensity: 4.4 or 9 kN/m^2

Live Load intensity: 2 kN/m^2

Floor Plan Area: 2025 m^2

Braced frame tributary area: 1012.5 m^2

Figure 3.6: Plan View of the Building with Two Braced Bays with BFTA of 1012.5 m^2



Dead Load intensity: 4.4 or 9 kN/m^2

Live Load intensity: 2 kN/m^2

Floor Plan Area: 2025 m^2

Braced frame tributary area: 506.25 m^2

Figure 3.7: Plan View of the Building with Four Braced Bays with BFTA of 506.25 m^2

A common side view for all plan types are given in the following figure:

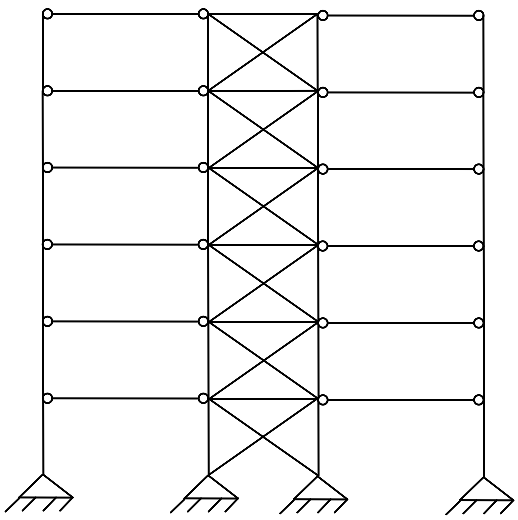


Figure 3.8: Side View of the Buildings

Braced widths of 3m, 4m, 5m, 6m, and 7m were considered for X-braced frames and 8m, 10m, 12m, and 14m for split-X braced frames. While the braced bay widths change the floor plan dimensions remain constant.

Four zones were considered in this study and these zones are identical to Turkish earthquake zones. The peak ground acceleration values for zones 1, 2, 3, and 4 are 0.4g, 0.3g, 0.2g, and 0.1g, respectively. For all zones 5 soil types described in Eurocode 8 were considered. Three importance classes (IC II, IC III, IC IV) that result in importance factors of 1.0, 1.2 and 1.4 were taken into account.

Two types of steel grades namely S235 and S355 were considered. The number of stories changed between 3 and 12 for X-braced frames and 3 and 16 for Split-X braced frames. In general the total height to braced bay width ratio was kept below 8.

European steel profiles were used in the design of braced frames. In general, all HEA, HEB, HD, HEM, IPN, IPE sections were included in the database. In addition, tubular sections produced by Borusan were added to the database. Typical designs of CBFs are given in the Appendix.

It has been argued in the past by Elghazouli (2009) that the overstrength rule given in Equation 2.22 is quite restrictive and can lead to uneconomical designs. Elghazouli (2009) proposed that using stiff columns through the height of the braced bay may be sufficient to omit the restriction placed on the overstrength of braces. In order to take into account this recommendation the overstrength provision was separately investigated. Designs were conducted by taking into account the provision given by Equation 2.22 as well as omitting it. The results for these designs are presented separately in the following sections.

3.2 Results of Parametric Study

The results of the parametric study are presented in this section. Results are presented separately depending on the type of design conducted (non-iterative vs. iterative), overstrength provision adopted, and importance factor. About 28800 buildings were considered for each analysis set. The results were compared with the initial period estimates and the lower bound proposed by Tremblay (2005).

3.2.1 Designs which Include Overstrength Provision

3.2.1.1 Importance Class II (γ_I=1.0)

The results for importance class II are given in Figures 3.9 through 3.12. In general, the lower bound expression given in Eurocode 8 is sufficient. There are some data points which fall below the lower bound expression proposed by Tremblay (2005). The data is quite scattered and some fundamental period values are significantly higher than the initial assumed period. In order to understand the level of conservatism the spectral accelerations obtained using the lower bound expression and the spectral accelerations obtained using the final period are also plotted. It is evident that the estimated design spectral accelerations are much higher compared to the actual design spectral accelerations. The differences are much more pronounced if an iterative type of design is conducted.

The upper bound equation proposed by Tremblay (2005) is sufficient to represent the designs conducted using the non-iterative method. There are only a few points above the upper bound line. On the other hand, there are more cases in the iterative design space which has periods higher than the periods estimated by the upper bound equation. Upper bound expressions obtained by curve fitting to the data developed in this study are given in Figures 3.9 and 3.10.

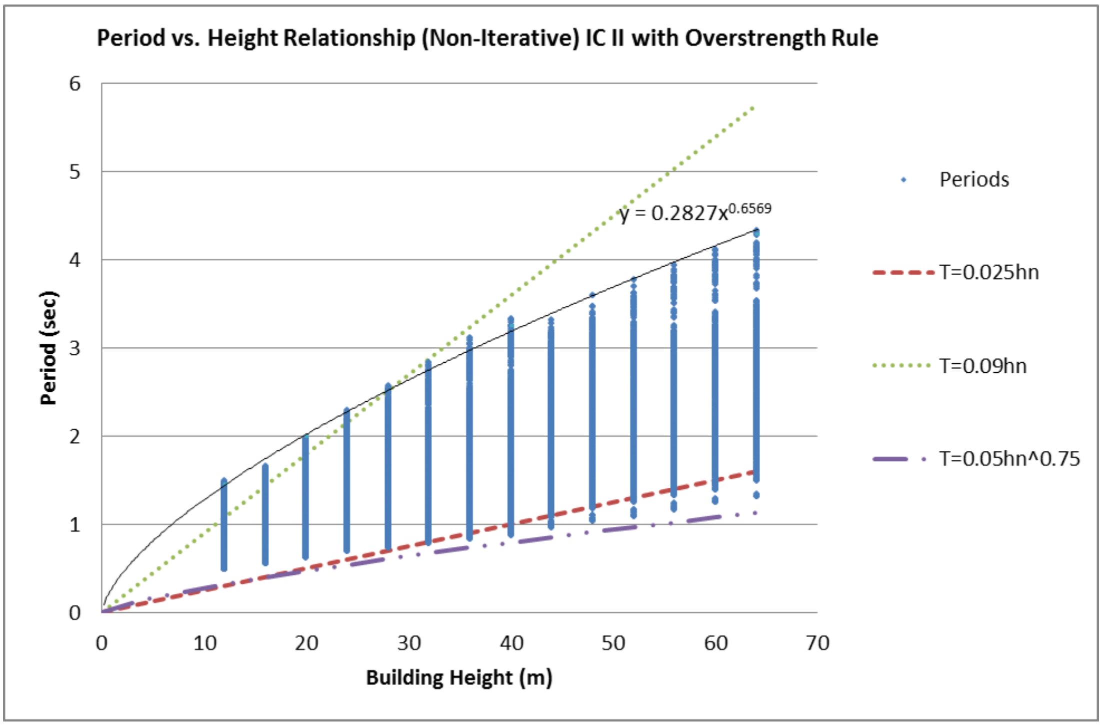


Figure 3.9: Period vs. Height Relationship – IC II – Non-Iterative with Overstrength Rule

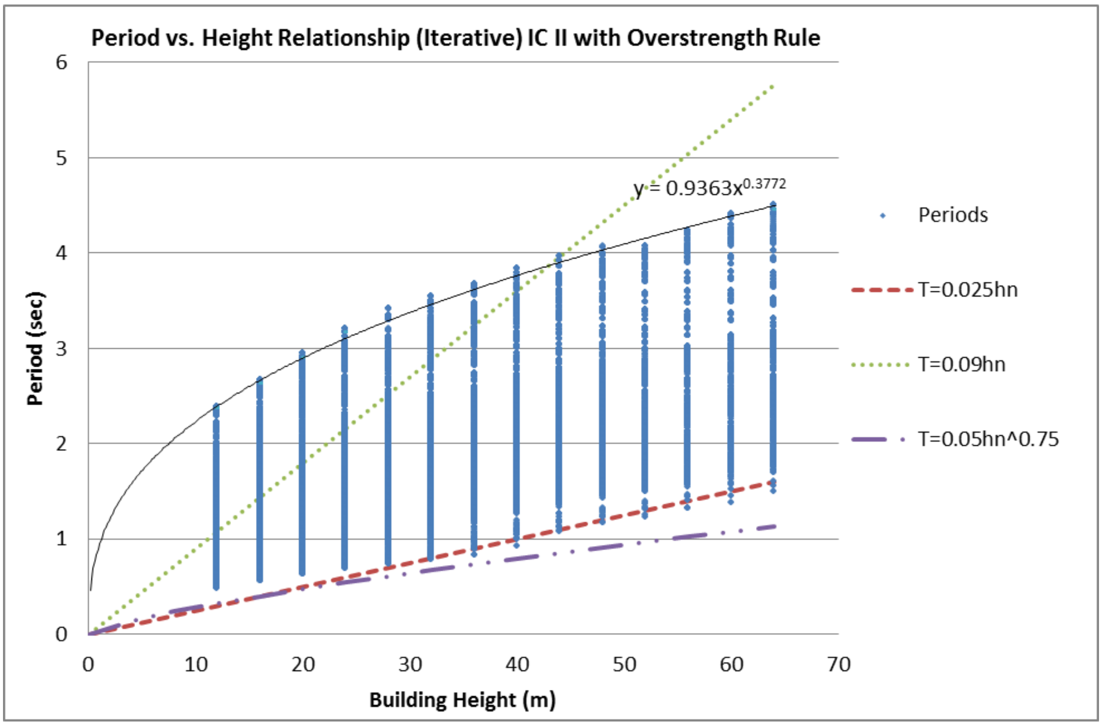


Figure 3.10: Period vs. Height Relationship – IC II – Iterative with Overstrength Rule

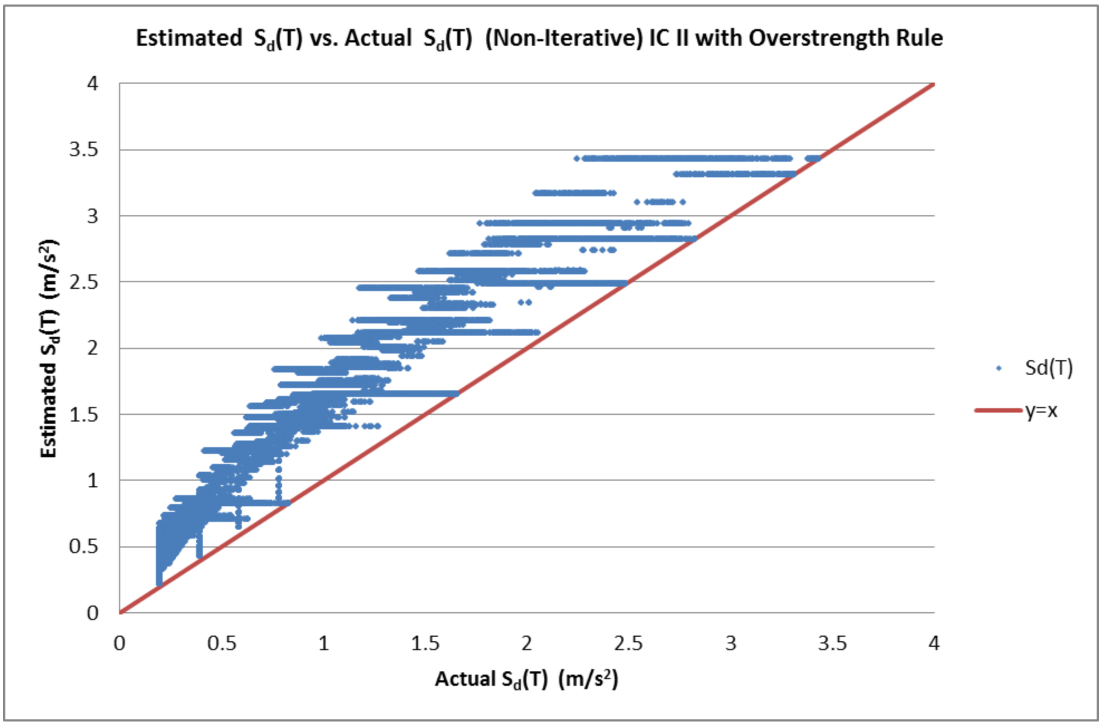


Figure 3.11: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC II – Non-Iterative with Overstrength Rule

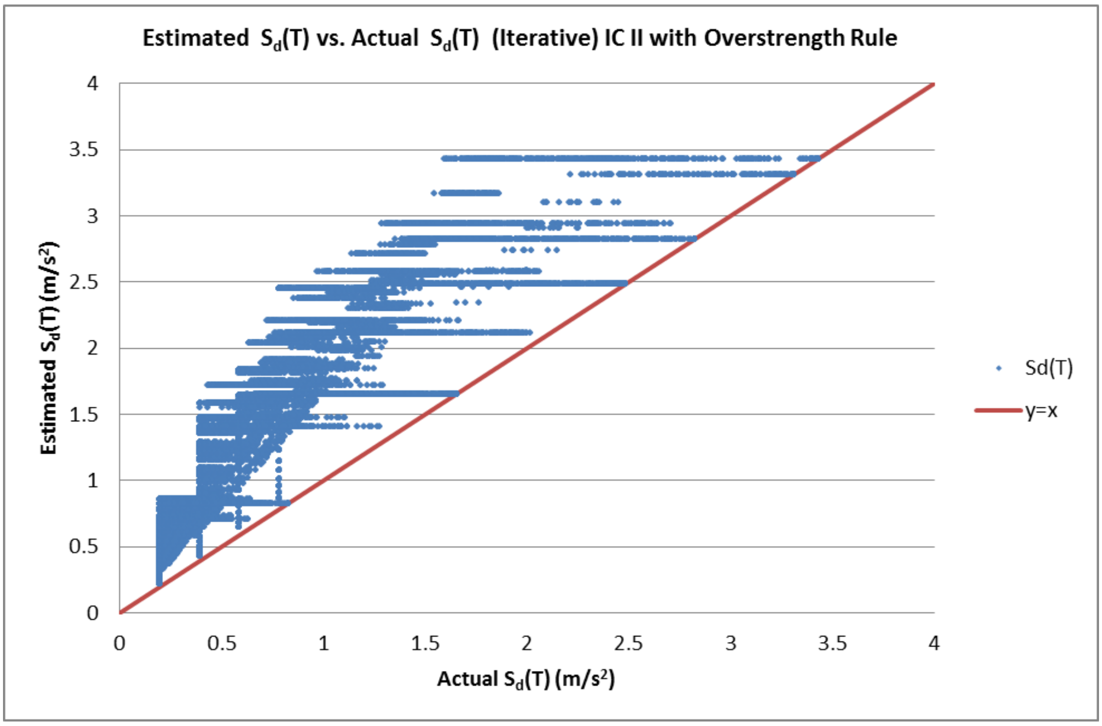


Figure 3.12: Estimated S_d(T) vs. Actual S_d(T) – IC II – Iterative with Overstrength Rule

The related statistics for the analysis cases is given in Table 3.1. According to the data given in this table the fundamental periods of the structures examined herein are on average 2.31 times higher than the periods obtained using the lower bound expression. This number modifies to 2.94 if an iterative type of design is conducted. Similarly the estimated design spectral accelerations are on average 1.85 and 2.22 times the actual design spectral accelerations for non-iterative and iterative design cases, respectively.

Table 3-1: Statistics of Data for Analysis of Importance Class II with Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Non-iterative)	Estimated $S_d(T)$ /Actual $S_d(T)$ (Iterative)
Mean	2.31	2.94	1.85	2.22
Standard Dev.	0.64	0.99	0.53	0.74
Max.	4.62	7.34	3.44	4.38
Min.	1.1	1.13	1	1

3.2.1.2 Importance Class III (γ_I=1.2)

The same cases were studied by considering Importance Class III. The results are presented in Figures 3.13 through 3.16 and the related statistics are given in Table 3.2. The very same conclusions can be derived for Importance Class III. The lower bound equation given by Eurocode 8 is sufficient to capture the response. There are a few points that fall below the lower bound expression developed by Tremblay (2005).

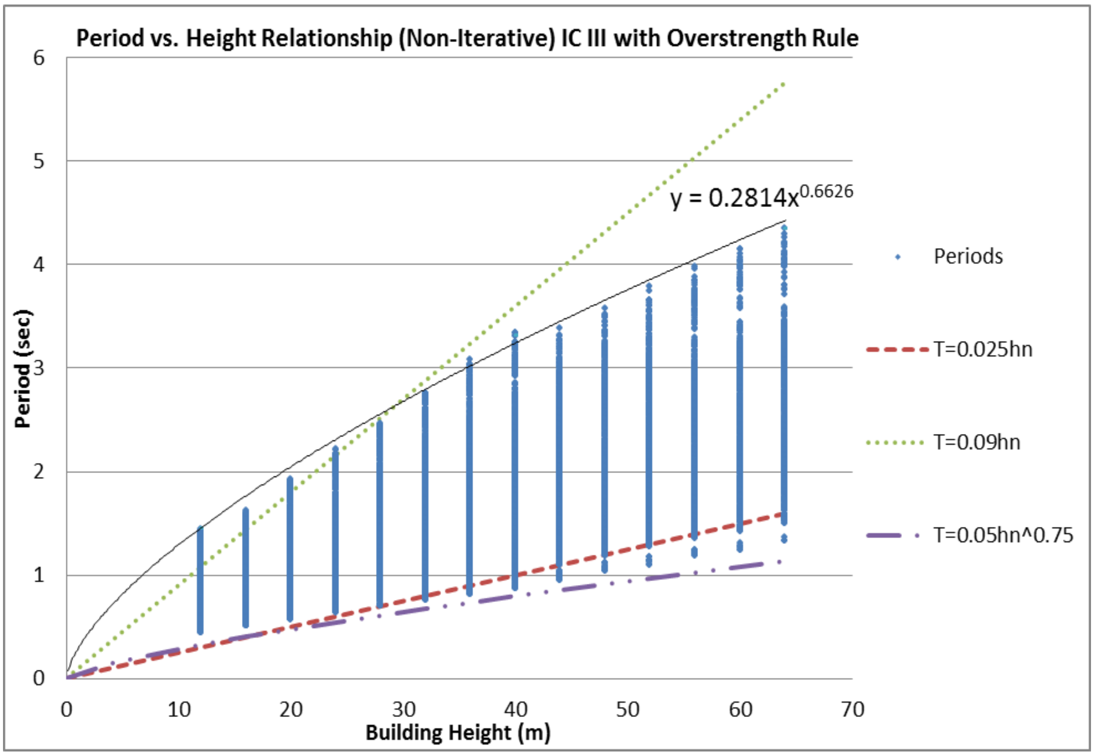


Figure 3.13: Period vs. Height Relationship – IC III – Non-Iterative with Overstrength Rule

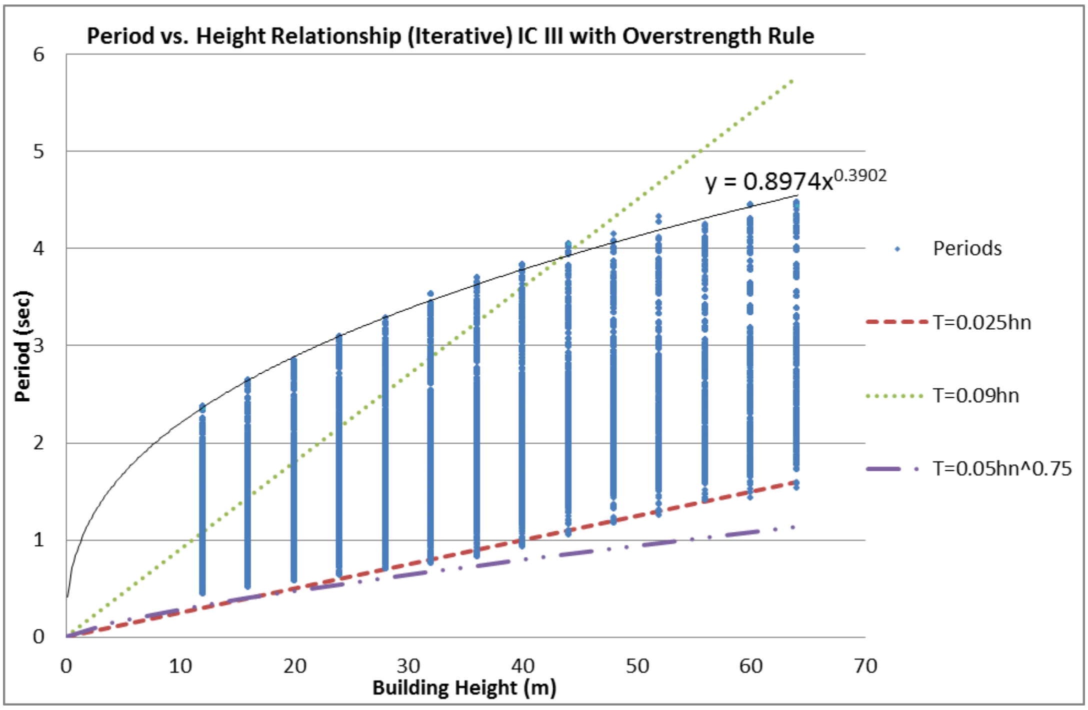


Figure 3.14: Period vs. Height Relationship – IC III – Iterative with Overstrength Rule

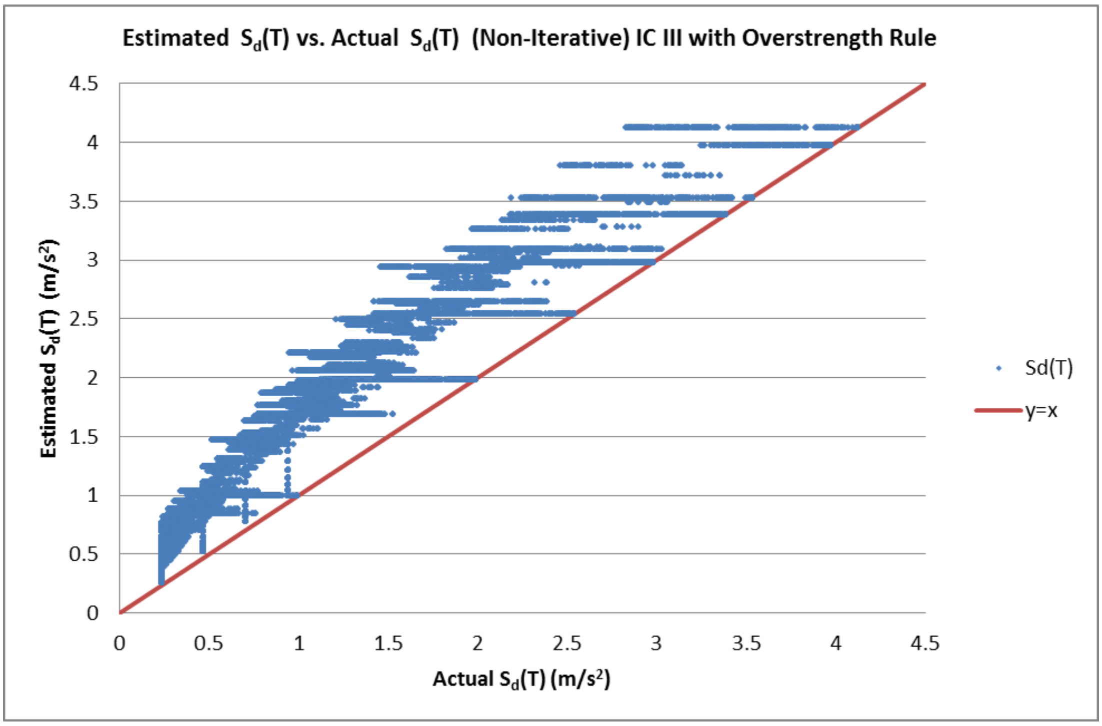


Figure 3.15: Estimated S_d(T) vs. Actual S_d(T) – IC III – Non-Iterative with Overstrength Rule

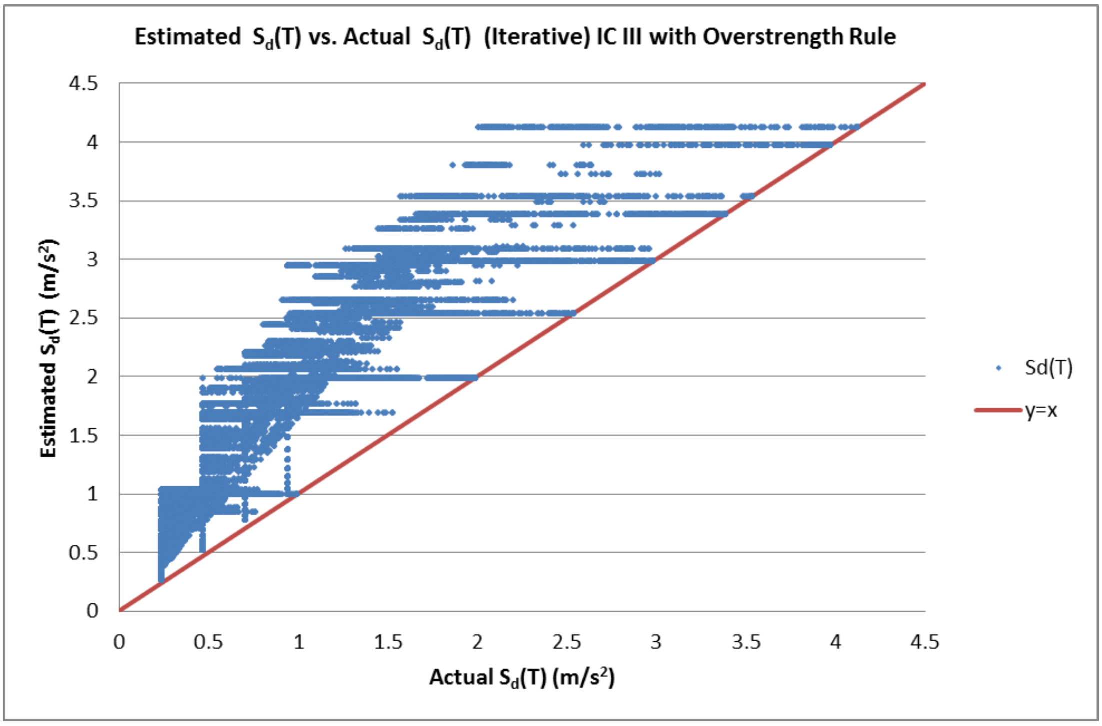


Figure 3.16: Estimated S_d(T) vs. Actual S_d(T) – IC III – Iterative with Overstrength Rule

Table 3-2: Statistics of Data for Analysis of Importance Class III with Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Non-iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Iterative)
Mean	2.25	2.86	1.82	2.18
Standard Dev.	0.62	0.97	0.52	0.74
Max.	4.48	7.36	3.31	4.38
Min.	1.09	1.12	1	1

3.2.1.3 Importance Class IV (γ_I=1.4)

The results for Importance Class IV are given in Figures 3.17 through 3.20 and the related statistics can be found in Table 3.3. The same conclusions can be derived for Importance Class IV.

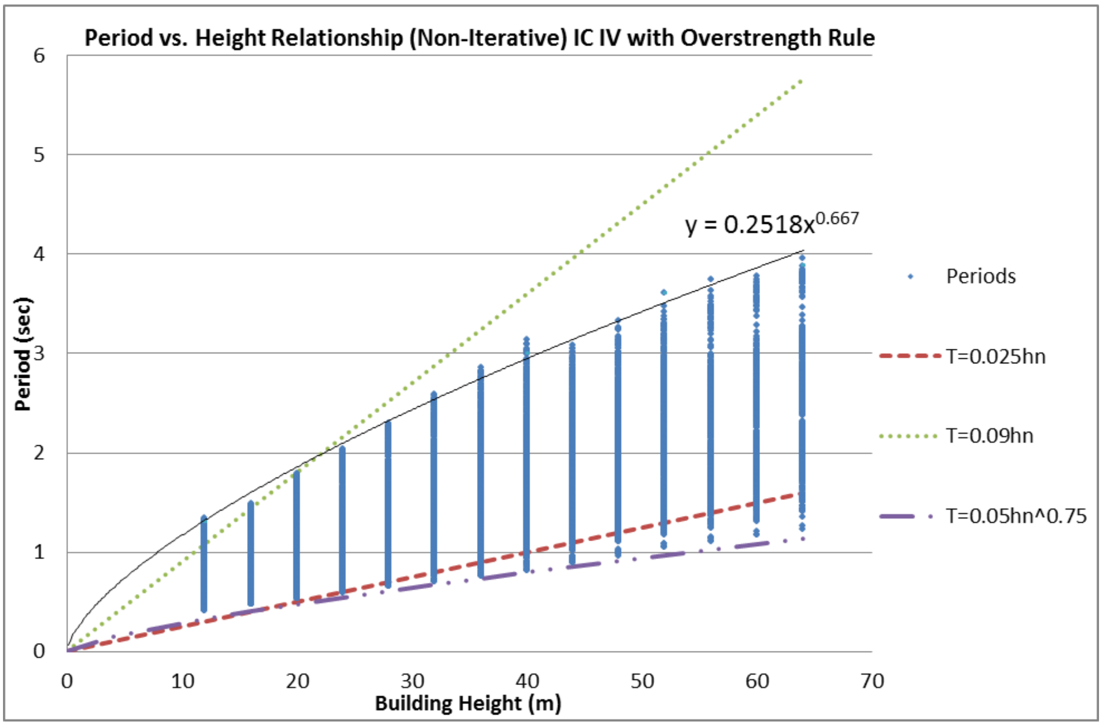


Figure 3.17: Period vs. Height Relationship – IC IV – Non-Iterative with Overstrength Rule

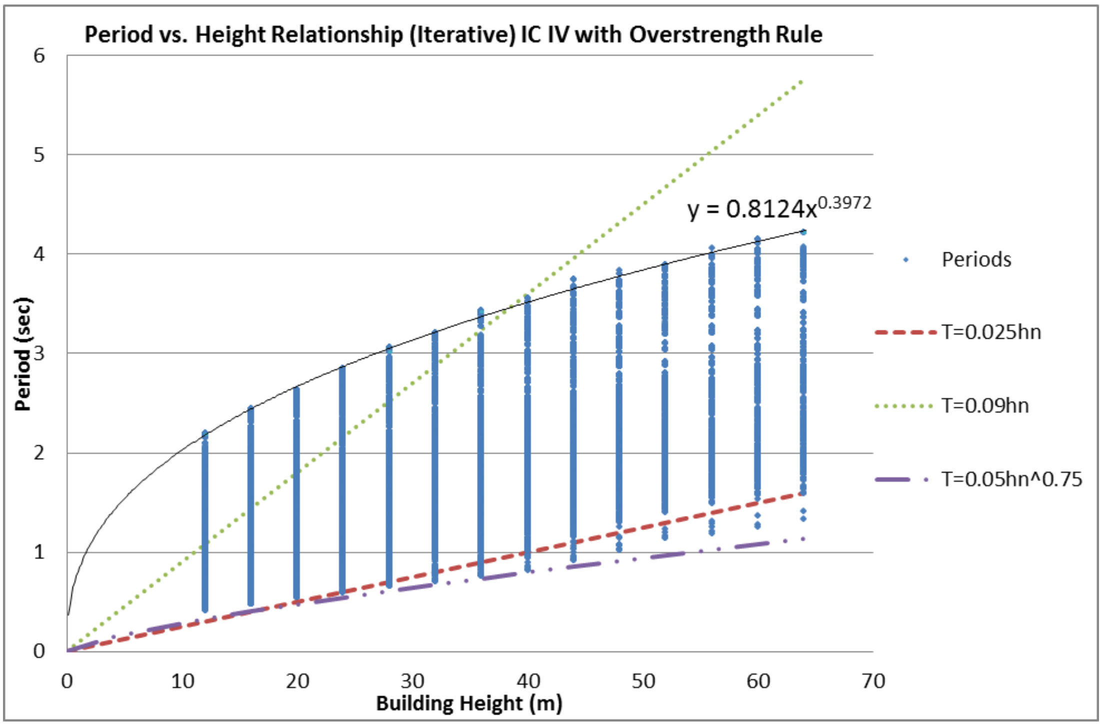


Figure 3.18: Period vs. Height Relationship – IC IV – Non-Iterative with Overstrength Rule

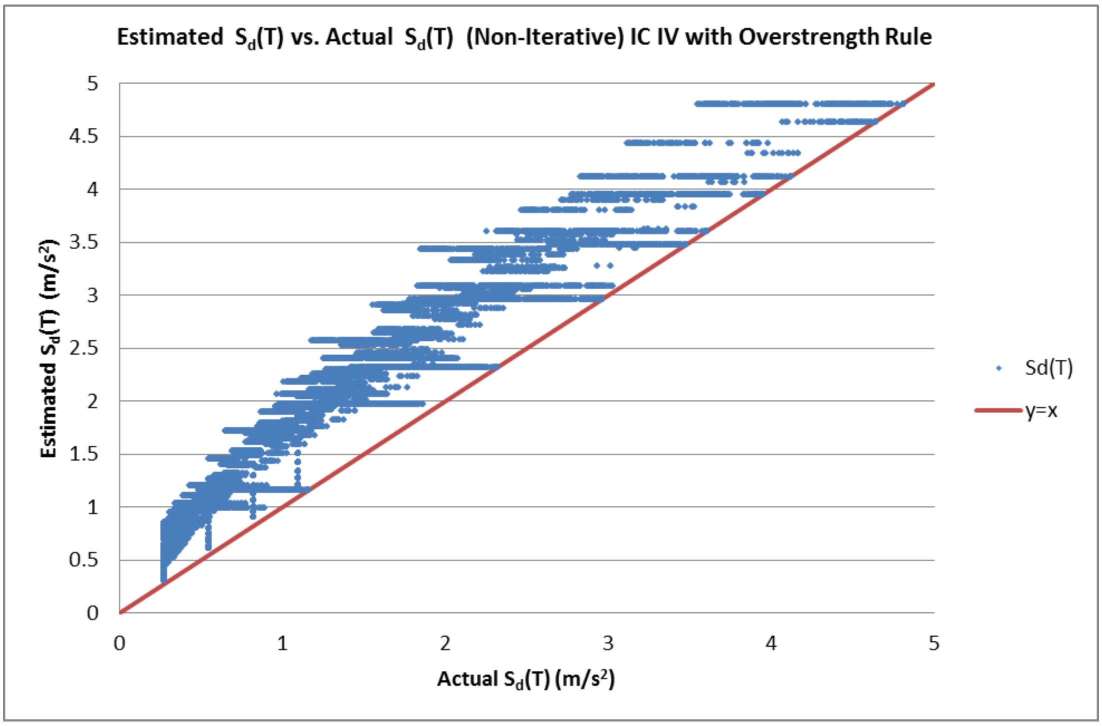


Figure 3.19: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC IV –Non-Iterative with Overstrength Rule

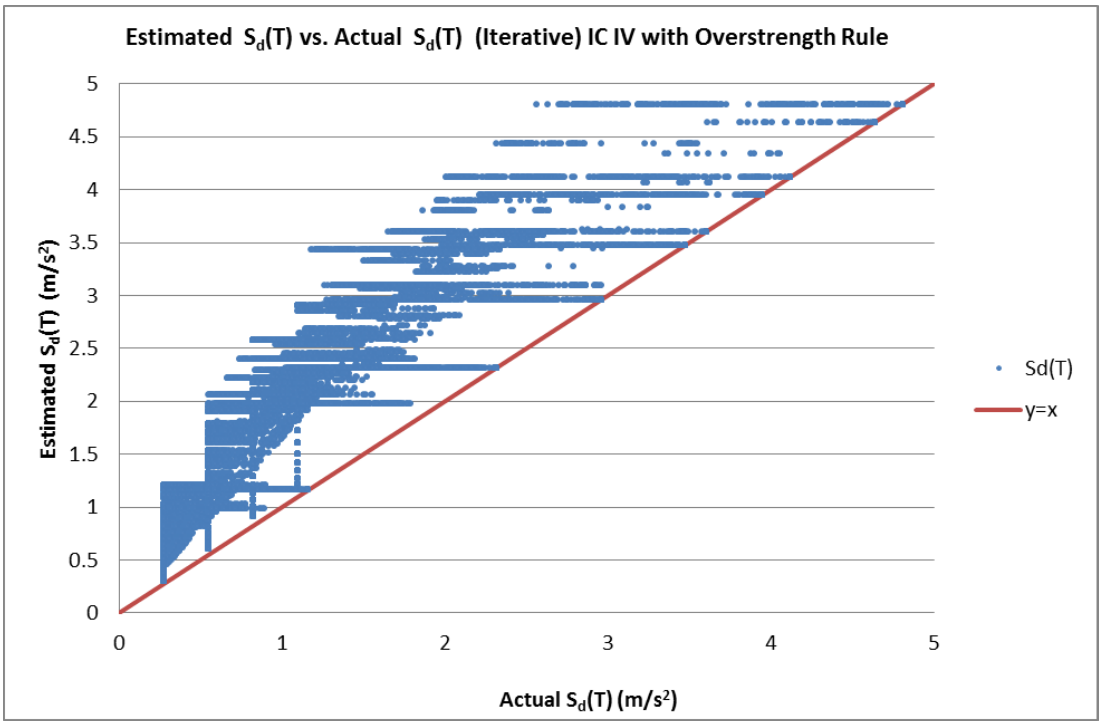


Figure 3.20: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC IV – Non-Iterative with Overstrength Rule

Table 3-3: Statistics of Data for Analysis of Importance Class IV with Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Non-iterative)	Estimated $S_d(T)$ /Actual $S_d(T)$ (Iterative)
Mean	2.14	2.71	1.74	2.097
Standard Dev.	0.58	0.93	0.50	0.73
Max.	4.18	6.83	3.13	4.38
Min.	1.02	1.03	1	1

3.2.2 Designs which Violate the Overstrength Rule

Same types of investigations were repeated to explore the effects of omitting the overstrength rule in the design of braced frames. The results were categorized according to the importance classes and are given in the following sections.

3.2.2.1 Importance Class II (γ_I=1.0)

The results for this category are given in Figures 3.21 through 3.24 and the related statistics can be found in Table 3.4. In general, omitting the overstrength rule does not have a significant effect on the period-height relationship. The statistical measures are quite similar to the ones obtained by including the overstrength provisions in design.

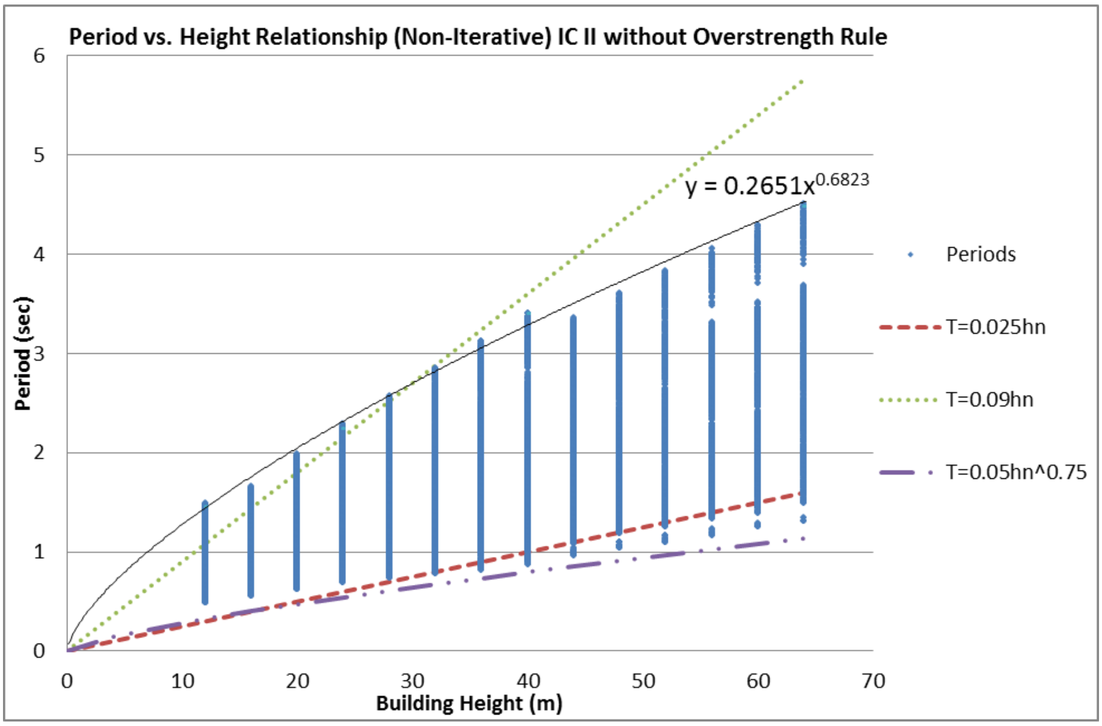


Figure 3.21: Period vs. Height Relationship – IC II – Non-Iterative without Overstrength Rule

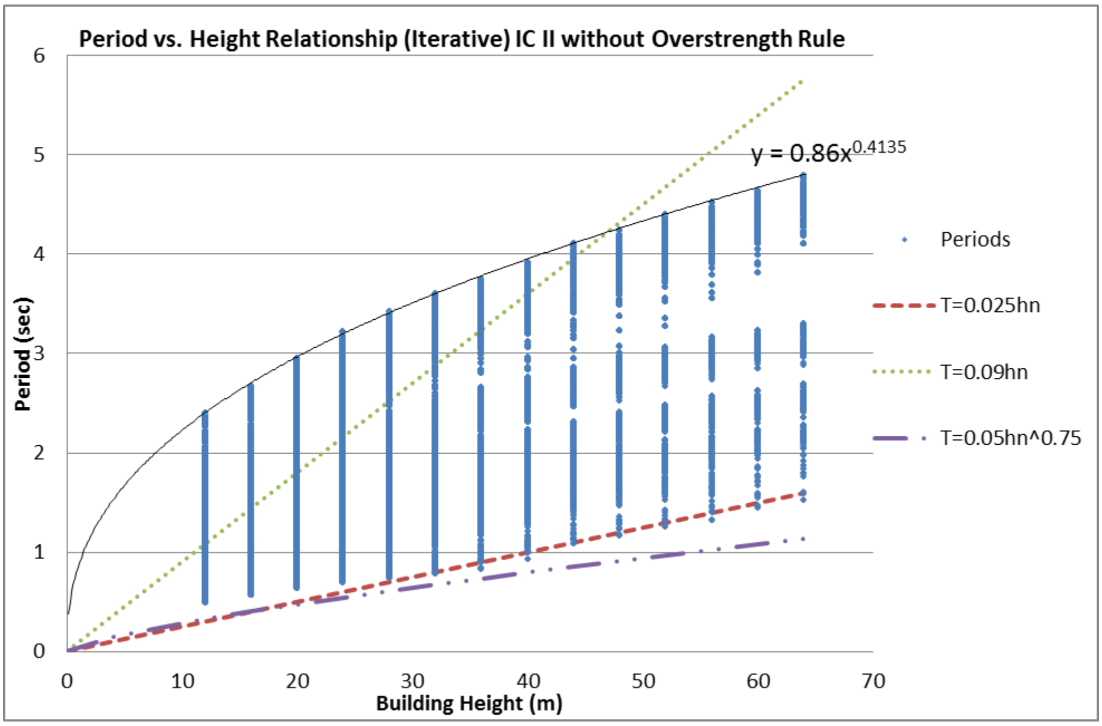


Figure 3.22: Period vs. Height Relationship – IC II – Iterative without Overstrength Rule

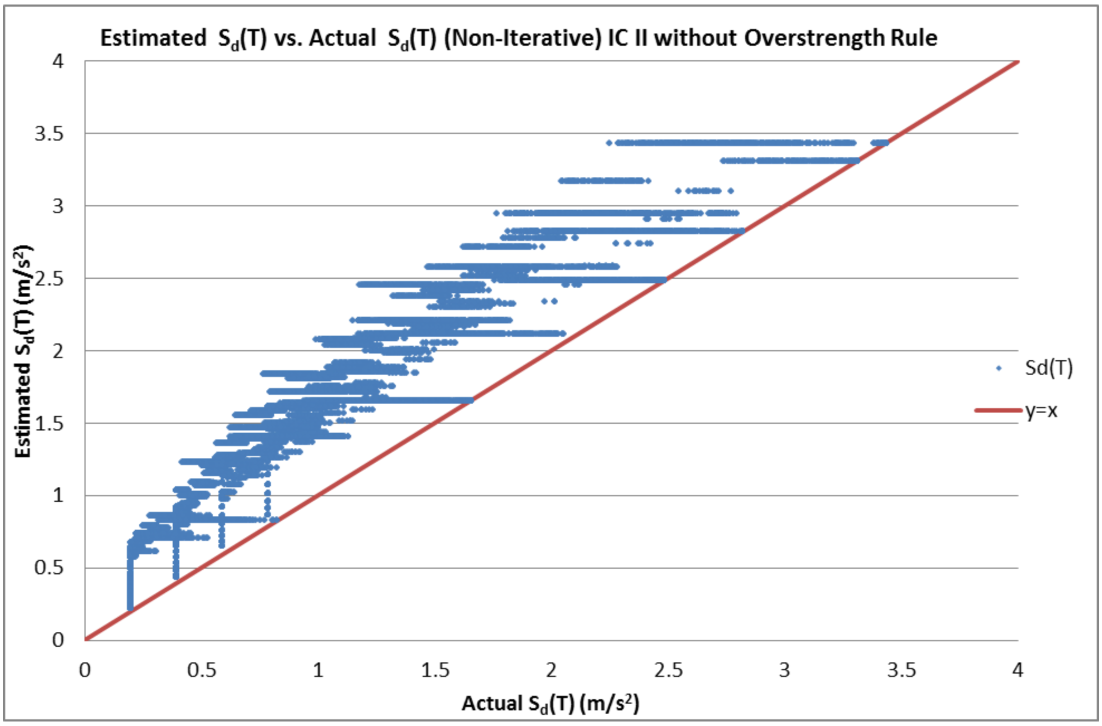


Figure 3.23: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC II –Non-Iterative without Overstrength Rule

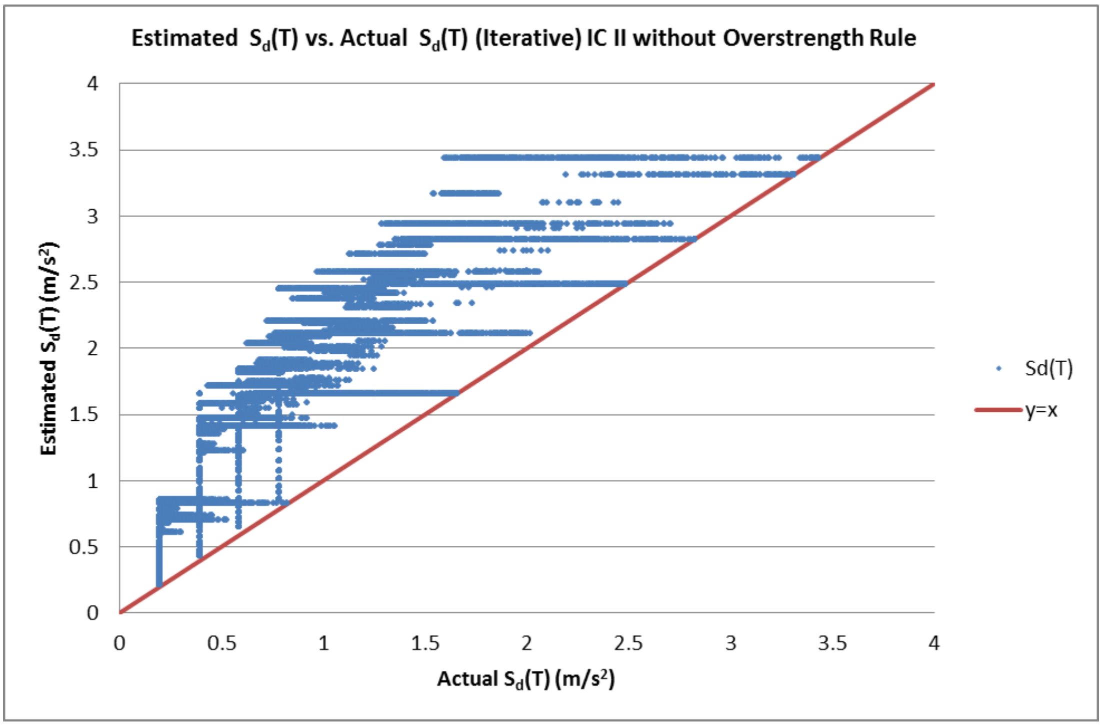


Figure 3.24: Estimated S_d(T) vs. Actual S_d(T) – IC II – Iterative without Overstrength Rule

Table 3-4: Statistics of Data for Analysis of Importance Class II without Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated S_d(T) / Actual S_d(T) (Non-iterative)	Estimated S_d(T) /Actual S_d(T) (Iterative)
Mean	2.40	3.37	1.88	2.35
Standard Dev.	0.70	1.15	0.55	0.83
Max.	4.62	7.44	3.44	4.38
Min.	1.10	1.13	1.00	1.00

3.2.2.2 Importance Class III (γ_I=1.2)

The results for this category are given in Figures 3.25 through 3.28 and the related statistics can be found in Table 3.5. In general, omitting the overstrength rule does not have a significant effect on the period-height relationship. The statistical measures are quite similar to the ones obtained by including the overstrength provisions in design.

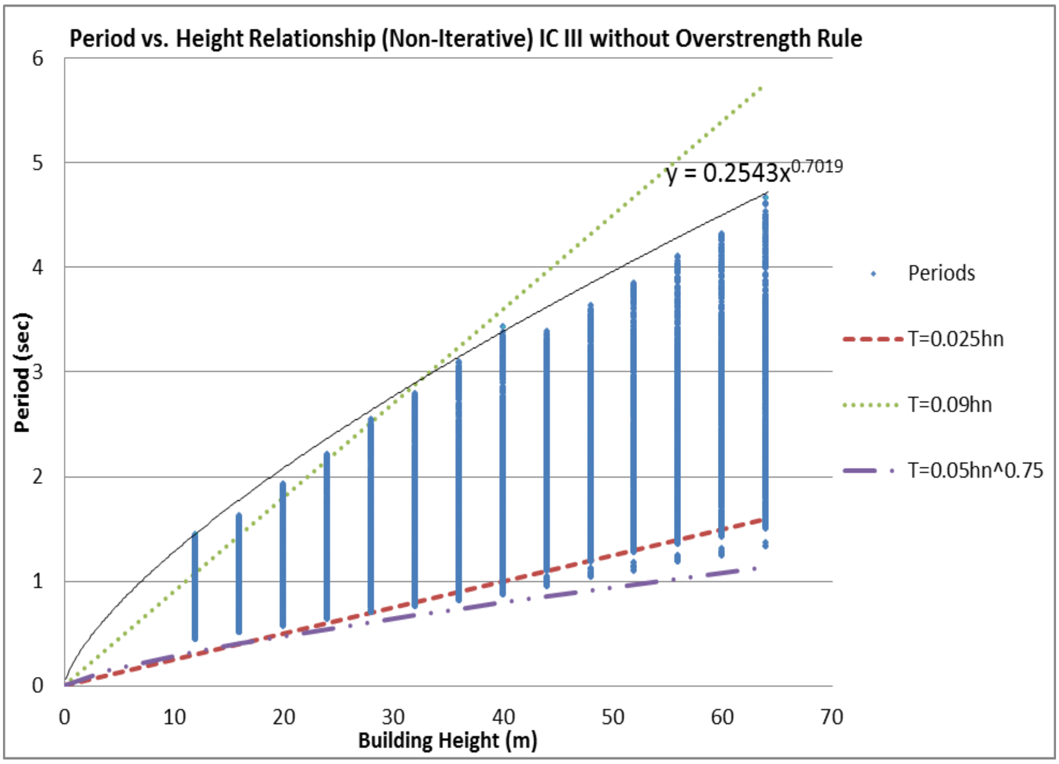


Figure 3.25: Period vs. Height Relationship – IC III – Non-Iterative without Overstrength Rule

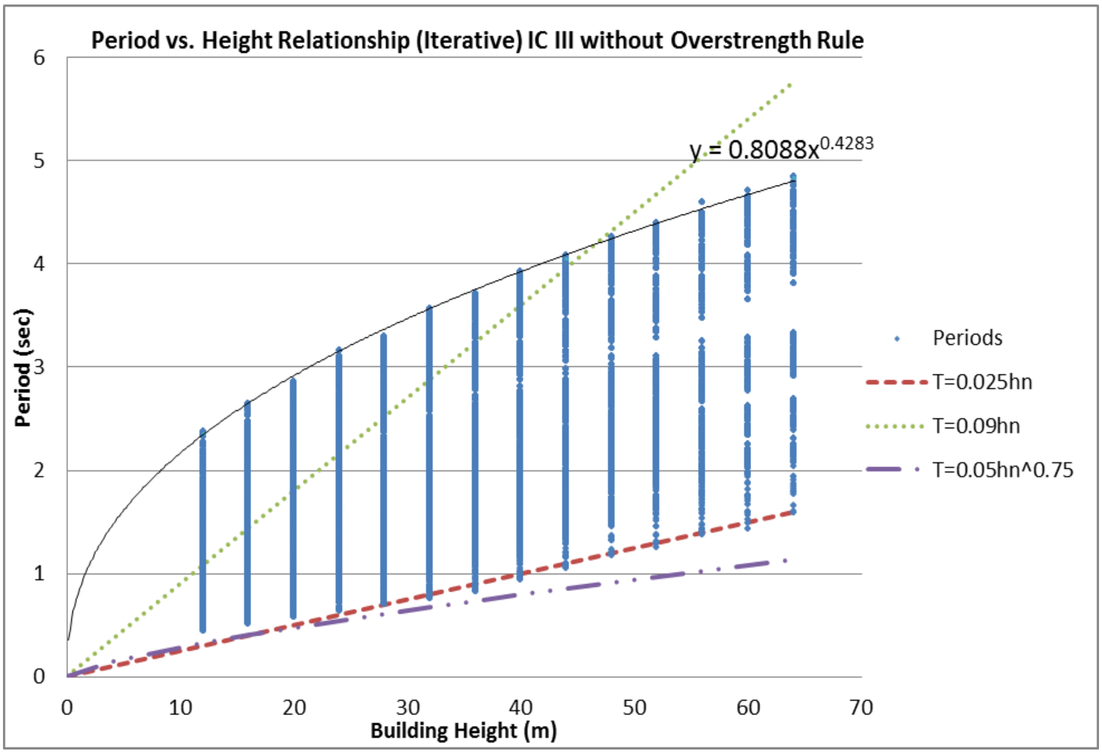


Figure 3.26: Period vs. Height Relationship – IC III – Iterative without Overstrength Rule

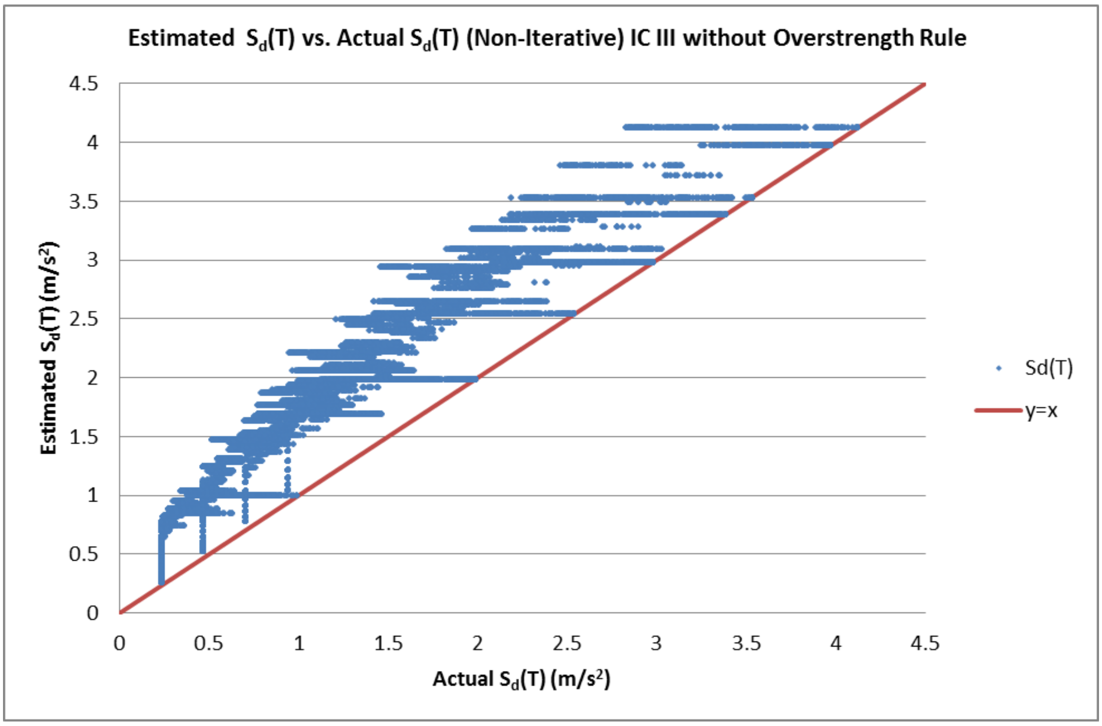


Figure 3.27: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC III – Non-Iterative without Overstrength Rule

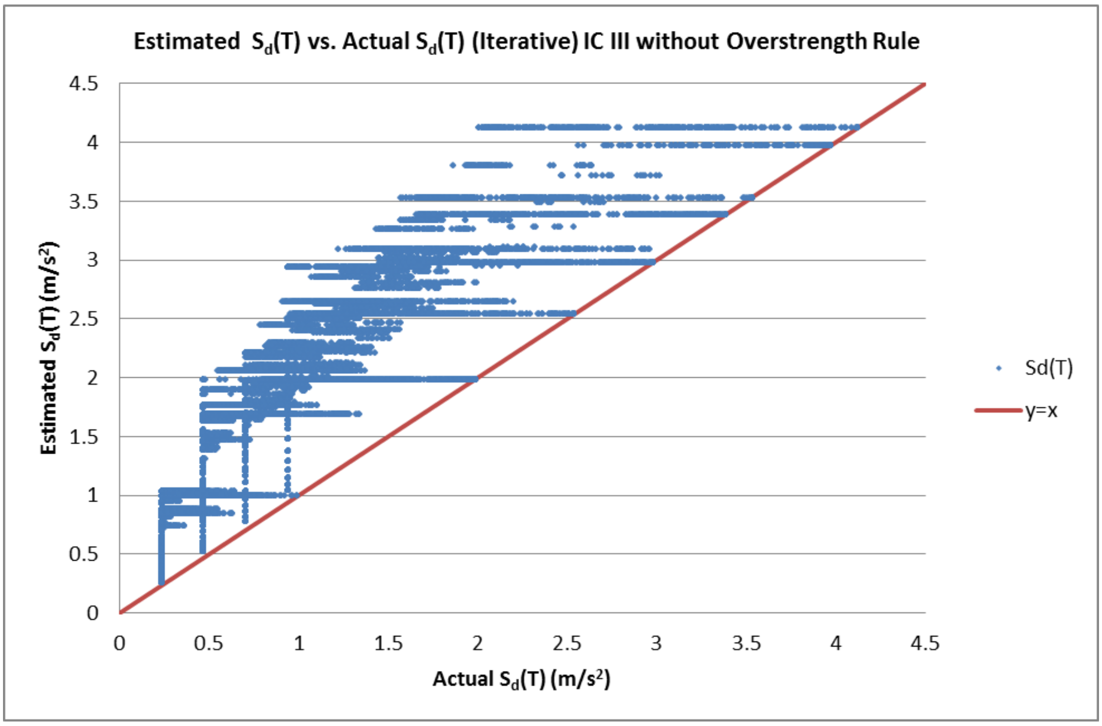


Figure 3.28: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC III – Iterative without Overstrength Rule

Table 3-5: Statistics of Data for Analysis of Importance Class III without Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Non-iterative)	Estimated $S_d(T)$ /Actual $S_d(T)$ (Iterative)
Mean	2.33	3.21	1.84	2.29
Standard Dev.	0.67	1.11	0.54	0.82
Max.	4.48	7.36	3.36	4.38
Min.	1.10	1.12	1.00	1.00

3.2.2.3 Importance Class IV (γ_I=1.4)

The results for this category are given in Figures 3.29 through 3.32 and the related statistics can be found in Table 3.6. In general, omitting the overstrength rule does not have a significant effect on the period-height relationship. The statistical measures are quite similar to the ones obtained by including the overstrength provisions in design.

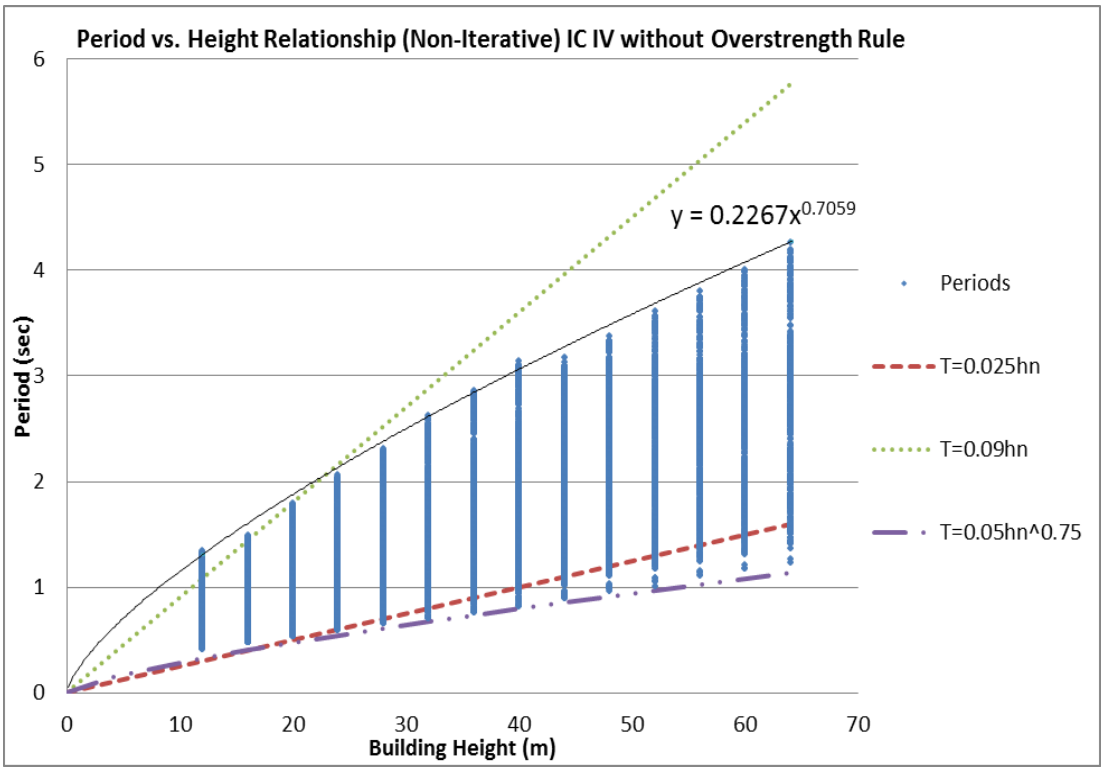


Figure 3.29: Period vs. Height Relationship – IC IV – Non-Iterative without Overstrength Rule

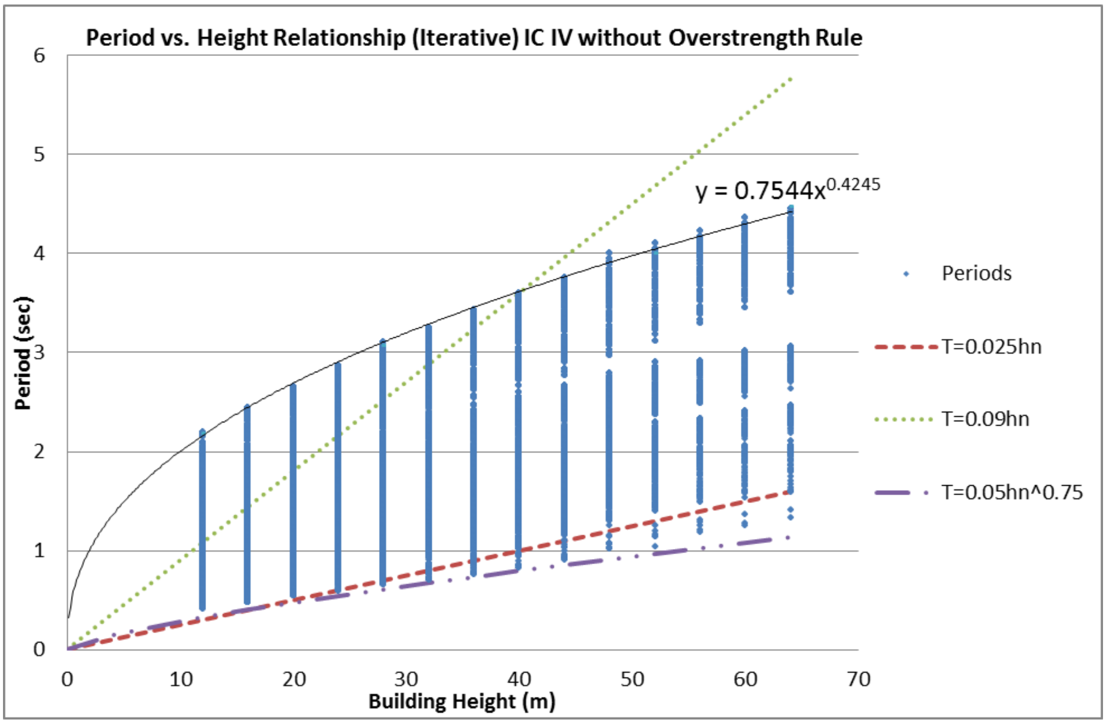


Figure 3.30: Period vs. Height Relationship – IC IV – Iterative without Overstrength Rule

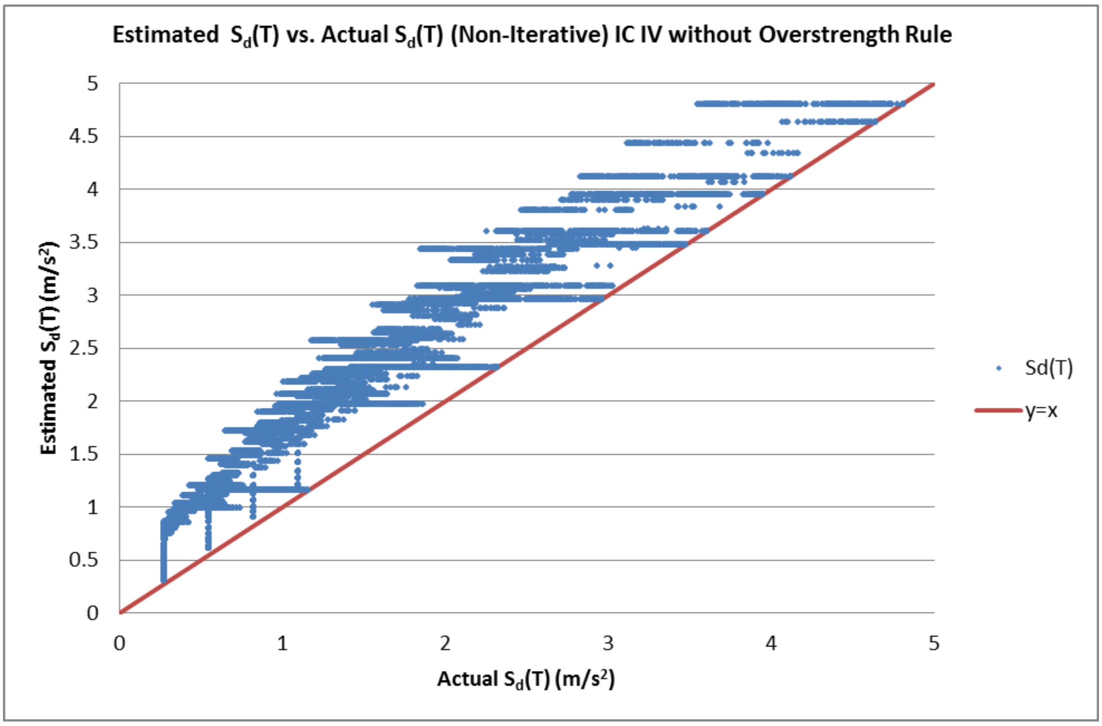


Figure 3.31: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC IV – Non-Iterative without Overstrength Rule

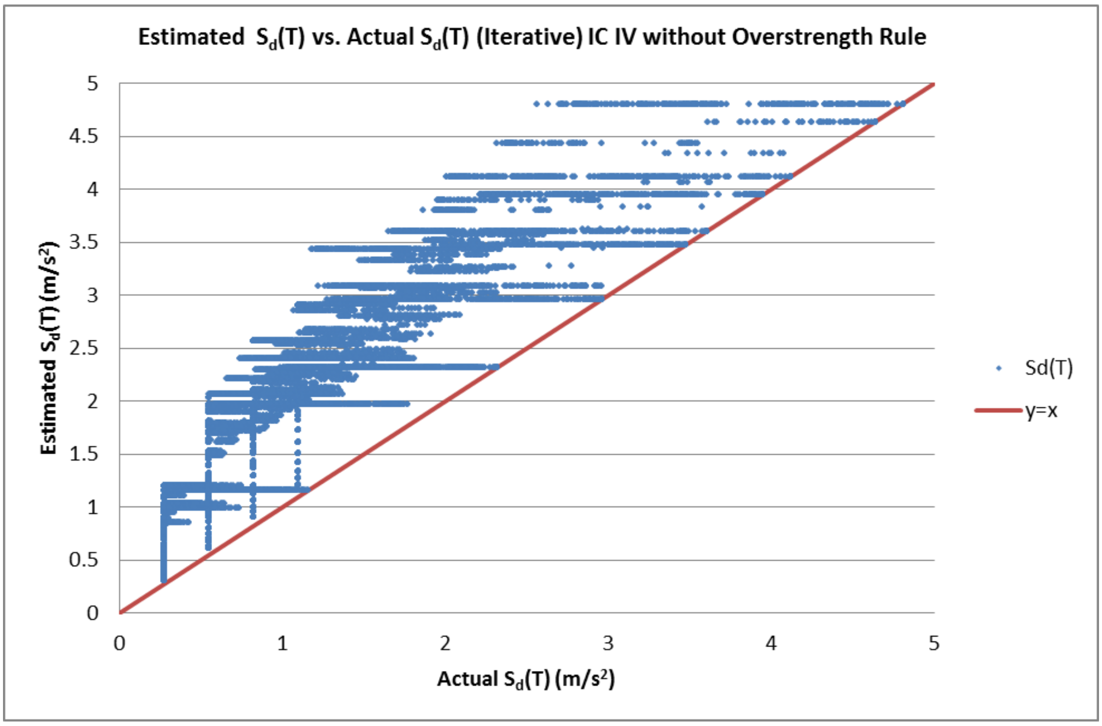


Figure 3.32: Estimated $S_d(T)$ vs. Actual $S_d(T)$ – IC IV – Iterative without Overstrength Rule

Table 3-6: Statistics of Data for Analysis of Importance Class IV without Overstrength Rule

Statistics	Actual period /Estimated period (Non-iterative)	Actual period /Estimated period (Iterative)	Estimated $S_d(T)$ / Actual $S_d(T)$ (Non-iterative)	Estimated $S_d(T)$ /Actual $S_d(T)$ (Iterative)
Mean	2.19	2.99	1.76	2.19
Standard Dev.	0.63	1.07	0.52	0.81
Max.	4.18	6.83	3.15	4.38
Min.	1.02	1.03	1.00	1.00

3.2.3 The Effect of Bracing Type and Seismic Level on Periods

It is seen that seismic level has an inversely proportional relationship with the calculated periods. In other words, periods go higher as the earthquake zone reduces. The mean values of all type of earthquake zones are shown in figure 3.33 and figure

3.34. As can be seen from these figures, the differences between the mean values become much more pronounced if an iterative type of design is conducted.

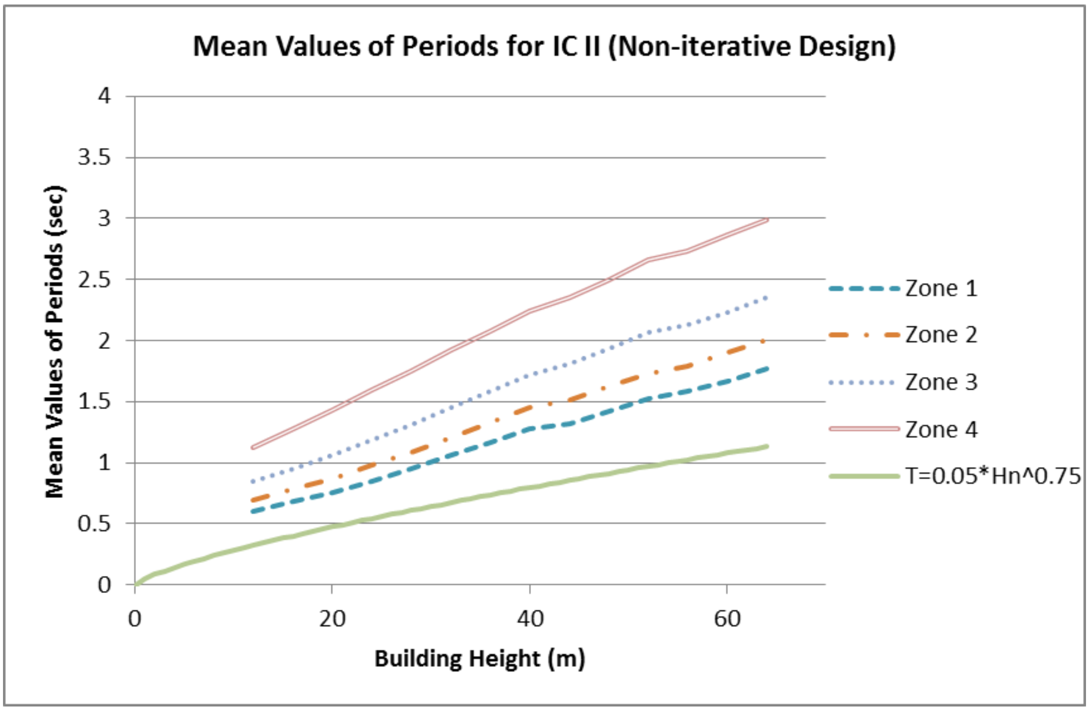


Figure 3.33: Mean Values of Periods for IC II (Non-Iterative Design)

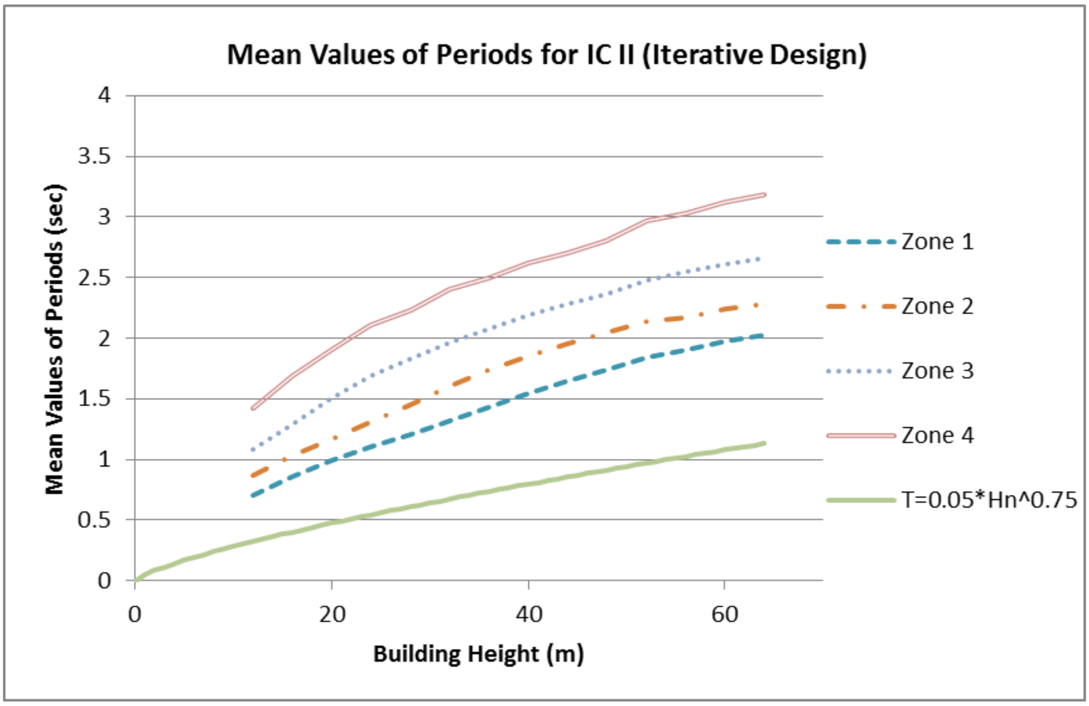


Figure 3.34: Mean Values of Periods for IC II (Iterative Design)

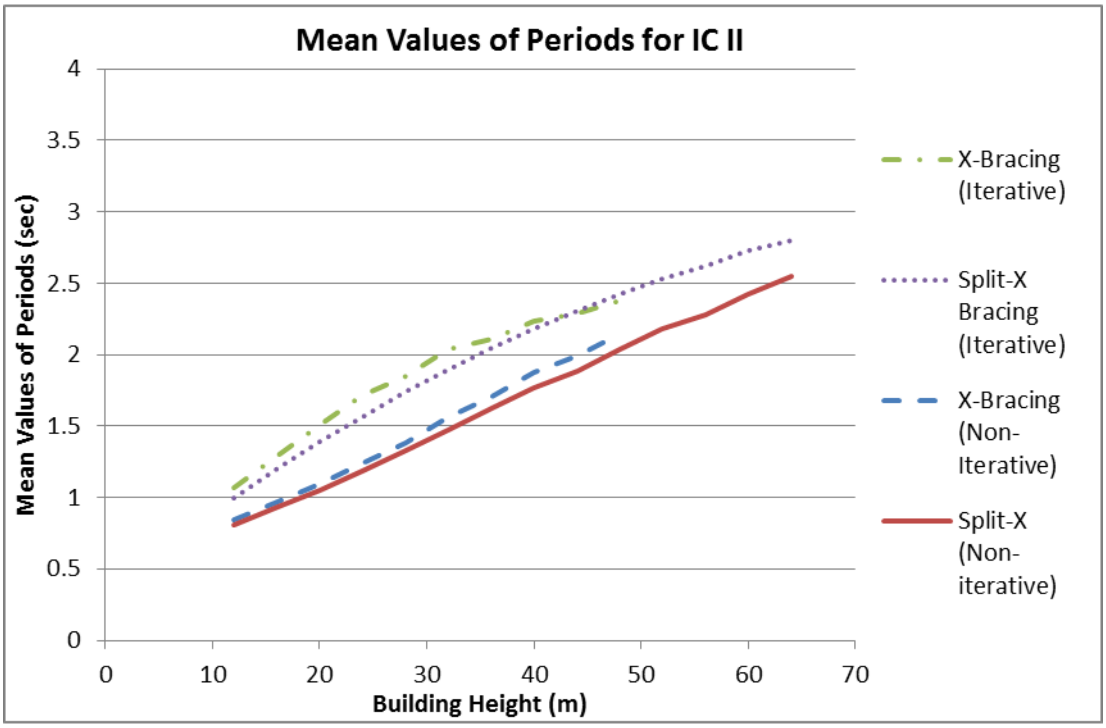


Figure 3.35: Mean Values of Periods according to Bracing Types for IC II

The two types of bracing system are found to be close each other. Namely, a significant difference was not observed between the X-bracing and Split-X bracing types.

3.2.4 Overstrength of Braced Frames

To show the differences between two different design philosophies the overstrength of the designed frames was investigated. The results are presented in Figures 3.36 and 3.37. In general the overstrength of frames is inversely proportional with the level of seismic action. Higher overstrengths are observed for cases with low seismicity and less reactive weight. For these cases the brace overstrength provisions given in Equation 2.22 governs the design and the resulting overstrengths can be quite high. In order to present the data in an effective manner the level of seismic action is represented by a variable ξ which is represented as follows:

$$\xi = \frac{a_g}{g} \times S \times \text{Braced Frame Tributary Area (m}^2) \times \text{ Story Mass Intensity (kN/m}^2) \quad (3.1)$$

Where;

$\frac{a_g}{g}$ and S are unitless.

This variable takes into account the peak ground acceleration, reactive weight per story, and soil conditions. According to Figure 3.36 there is a decreasing trend with ξ and the overstrength values reach to 10 which is illogical because the overstrength value should not be greater than the behavior factor. Much more meaningful results are obtained when the overstrength provision is omitted. In this case the maximum overstrength values reach to 4.5 and majority of the values are below 2.0. This aspect of design should be considered by the developers of Eurocode 8. As explained by Elghazouli (2009) the use of the overstrength provision can sometimes be overly restrictive.

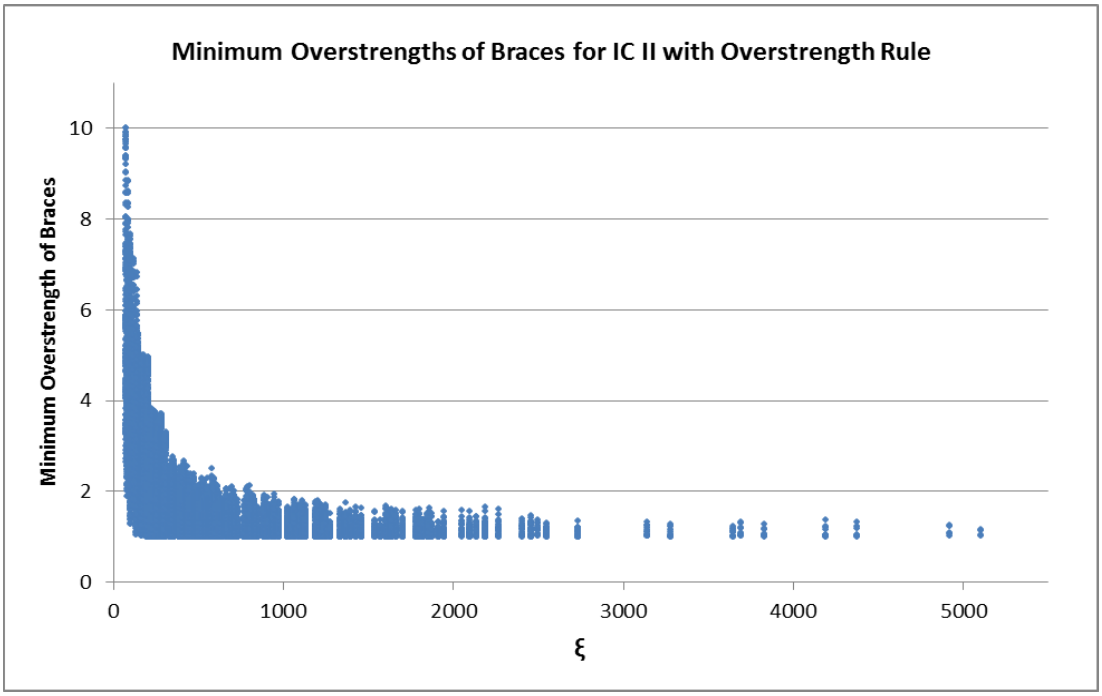


Figure 3.36: Minimum Overstrengths of Braces for IC II with Overstrength Rule

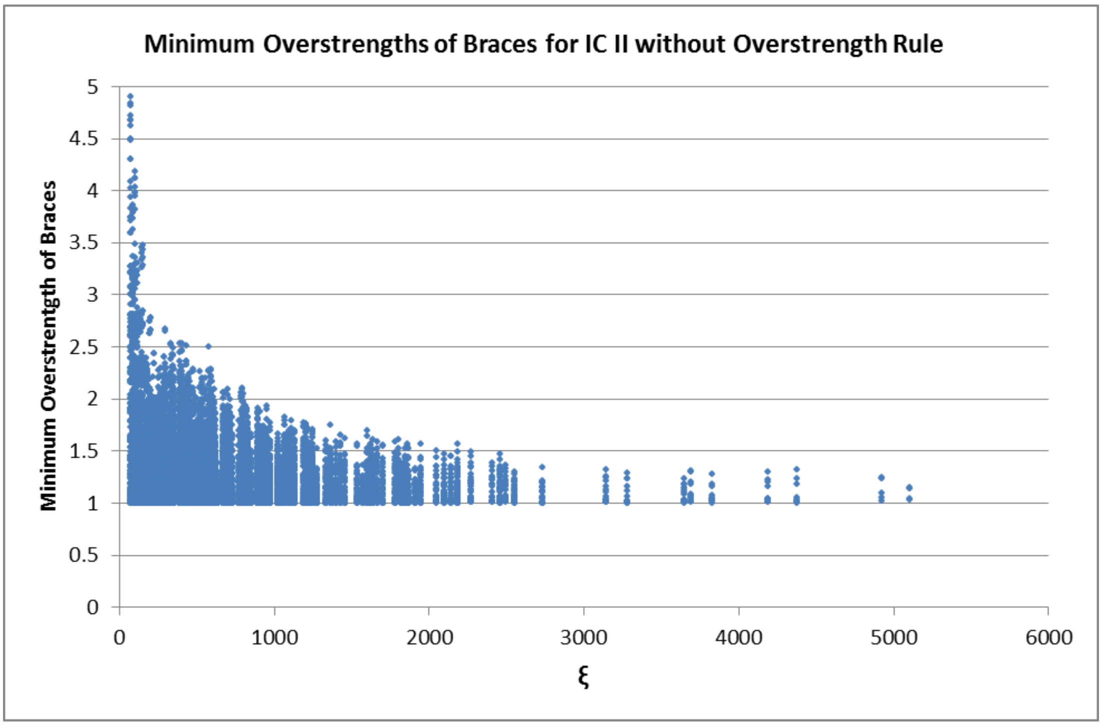


Figure 3.37: Minimum Overstrengths of Braces for IC II without Overstrength Rule

CHAPTER 4

DEVELOPMENT OF AN EMPIRICAL FORMULA FOR DETERMINING THE FUNDAMENTAL PERIOD OF BRACED FRAMES

The results presented in Chapter 3 revealed that the natural period expressions provided in the design specifications are sufficient as a lower bound. These can result in overly conservative estimates of the design spectral acceleration. In this Chapter an alternative method is developed to predict the fundamental natural period of braced fames with more accuracy. The objective of this derivation is to reduce the amount of scatter in predicting the periods using lower bound expressions. In this chapter the detailed derivation of the method is presented followed by its verification.

4.1 Development of an Alternative Formula for Estimating Fundamental Period of Braced Frames

This method requires the top story drift ratio (ζ) to estimate the fundamental natural period. This information is derived from the design set developed in this thesis. The following assumptions are adopted during the development of this method:

- All stories have equal mass. In other words, the method is applicable to structures having the same mass properties at all floors. This is a valid assumption for most of the residential and office type buildings. Although the roof level may contain a smaller amount of mass compared to other stories, this difference does not lead to significant errors.
- Story height is constant along the height of the building. In other words, the distance between the floor levels is the same. For most of the regular

residential and office buildings this assumption is valid. Although in some structures the height of the first story can be greater than the others it is considered that larger first story height does not lead to significant errors.

- The equal mass and story height assumptions lead to an inverted triangular type of load pattern according to the equivalent lateral force procedure described in Eurocode 8.
- Lateral displacements of the stories vary linearly over the height of the building as shown in Figure 4.1.

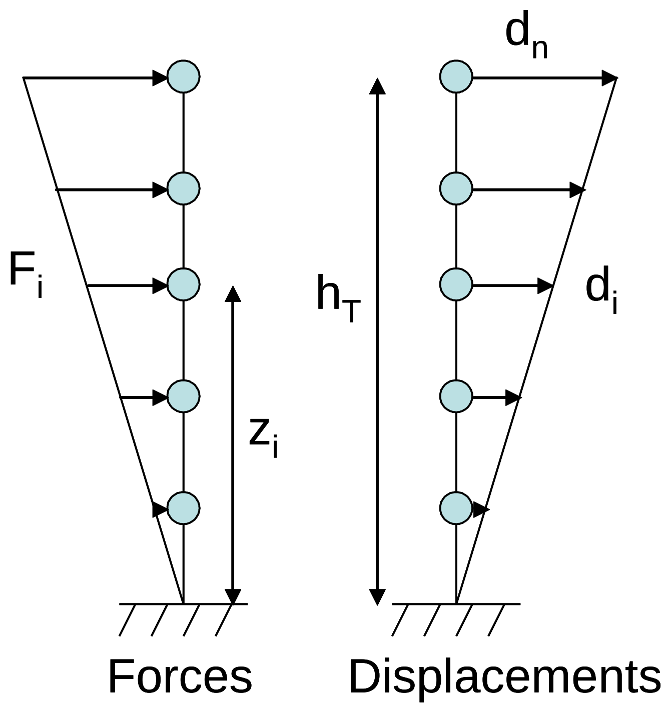


Figure 4.1: Representative Force and Displacement Pattern

- It was explained by Goel and Chopra (1997) that the period of a structure under similar assumptions can be expressed as follows:

$$T = C_3 H^{1/2-\gamma} \tag{4.1}$$

where; C_3: constant that depends on the type of lateral load resisting system, H: total height of the building, and γ: coefficient that represents the response spectrum where the base shear is proportional to $1/T^{\gamma}$. It is evident from Equation 4.1 that if the coefficient γ is equal to 2 then the resulting solution is undefined. Therefore, in the derivation of the present method the design response spectrum has been modified to eliminate the region defined by $T>T_D$. Instead the spectrum function recommended for the region $T_C \leq T \leq T_D$ is used for periods $T_C \leq T$. This assumption results in conservative values for design base shear. A comparison of the original and modified spectra is shown in Figure 4.2. As shown in this figure omitting the region defined by $T>T_D$ slightly modifies the spectrum and for most of the soil types the minimum base shear governs the design. Therefore, only one function which is represented in Equation 4.2 is considered to represent the response spectrum for the range $T_C \leq T$.

$$S_d(T) = a_g \times S \times \frac{2.5}{q} \times \left[\frac{T_c}{T}\right] \tag{4.2}$$

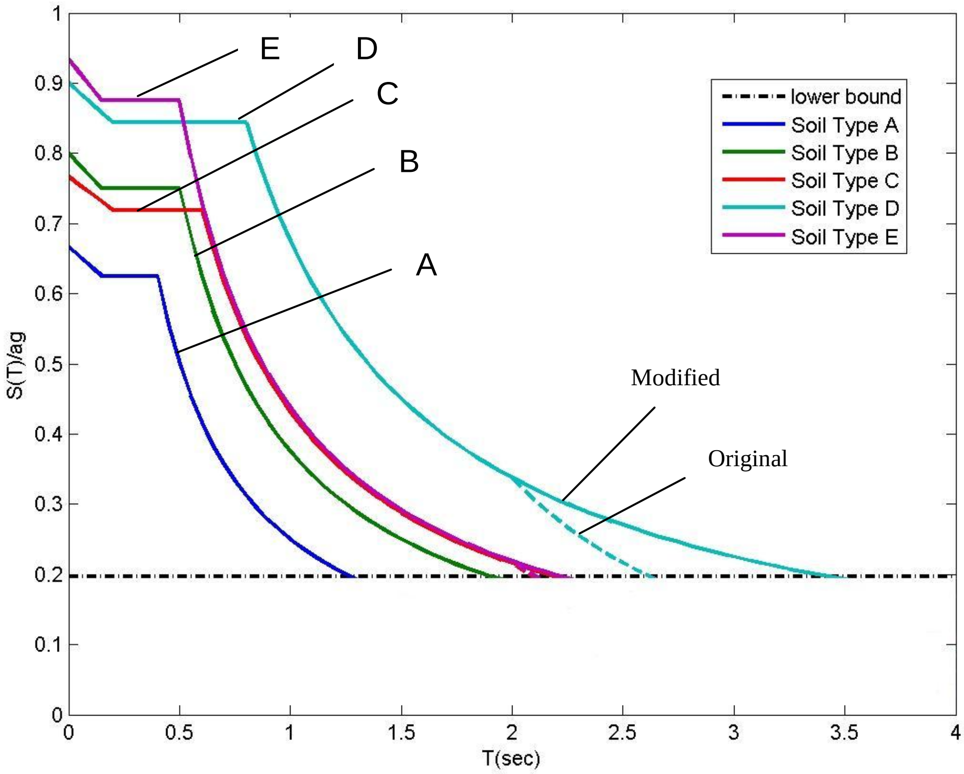


Figure 4.2: A Comparison of the Original and Modified Spectra for q=4

Under these assumptions the fundamental period of a structure can be calculated using the Rayleigh's method as follows:

$$T = 2 \times \pi \times \sqrt{\frac{\sum_{i=1}^{N} m_i d_i^2}{\sum_{i=1}^{N} F_i d_i}} \tag{4.3}$$

Where m_i: mass of the i^th^ story which is assumed to be equal to m for all stories, F_i: lateral force at level i, d_i: displacement at the i^th^ story, N: number of stories.

According to Eurocode 8 the lateral force at each story is equal to:

$$F_i = F_b \frac{z_i m_i}{\sum_{j=1}^{N} z_j m_j} \tag{4.4}$$

$$F_b = \frac{C_1}{T} N \times m \tag{4.5}$$

$$C_1 = \lambda \times \delta \times a_g \times S \times \frac{2.5}{q} \times T_c \tag{4.6}$$

Where z_i: is the height of the i^th^ story measured from the base.

The displacement of each story can be expressed in terms of the top story drift ratio as follows:

$$d_i = \varsigma \times z_i \tag{4.7}$$

Inserting Equations 4.4, 4.5, 4.6 and 4.7 into Equation 4.3 yields:

$$T = 2 \times \pi \times \sqrt{\frac{m \sum_{i=1}^{N} \left(\varsigma^2 \times z_i^2 \right)}{\sum_{i=1}^{N} \left(\frac{C_1}{T} \times \frac{z_i \times m}{m \sum_{j=1}^{N} z_j} \times N \times m \times \varsigma \times z_i \right)}} \tag{4.8}$$

Equation 4.8 simplifies to:

$$T = 2 \times \pi \times \sqrt{\frac{m \times \varsigma^2 \sum_{i=1}^{N} z_i^2}{\frac{C_1 \times N \times m \times \varsigma}{T} \times \sum_{i=1}^{N} \frac{z_i^2}{\sum_{j=1}^{N} z_j}}} \tag{4.9}$$

Further simplification of Equation 4.9 yields

$$T = 2 \times \pi \times \sqrt{\frac{T \times \varsigma}{C_1 \times N} \sum_{j=1}^{N} z_j} = \frac{4 \times \pi^2 \times \varsigma}{C_1 \times N} \sum_{j=1}^{N} z_j \tag{4.10}$$

For buildings with equal story height, the z_j factor can be expressed as follows:

$$z_j = j \times h_s \tag{4.11}$$

Where h_s: height of one story.

Equation 4.10 can be rewritten by making use of Equation 4.11 as follows:

$$T = \frac{4 \times \pi^2 \times \varsigma \times h_s}{C_1 \times N} \sum_{j=1}^{N} j \tag{4.12}$$

The summation term in Equation 4.12 can be written in terms of the number of stories as follows:

$$\sum_{j=1}^{N} j = \frac{N(N+1)}{2} \tag{4.13}$$

Inserting Equation 4.13 into Equation 4.12 yields:

$$T = \frac{2\times\pi^2\times\varsigma\times h_s\times(N+1)}{C_1} = \frac{2\times\pi^2\times q\times\varsigma\times h_s\times(N+1)}{2.5\times\lambda\times\delta\times a_g\times S\times T_C} \tag{4.14}$$

Denoting the term $q\times\varsigma$ as the inelastic top story drift ratio ζ_I, Equation 4.14 can be expressed as follows:

$$T = \frac{2\times\pi^2\times h_s\times(N+1)}{2.5\times\lambda\times\delta\times a_g\times S\times T_C}\times\varsigma_I \tag{4.15}$$

Equation 4.15 can be used to estimate the fundamental natural period of a structure if the inelastic top story drift ratio is known in advance. It should be noted that the developed expression is valid in the range $T_C \leq T$. The developed period estimation equation can be used to calculate the design spectral accelerations. If Equation 4.15 is inserted into Equation 4.2 the following expression can be derived for the design spectral acceleration:

$$S_d(T) = \left(\frac{2.5\times a_g\times S\times T_C}{\pi}\right)^2 \frac{\lambda\times\delta}{2\times q\times h_s\times(N+1)\times\varsigma_I} \tag{4.16}$$

Where $a_g\times S\times\frac{2.5}{q} \geq S_d(T) \geq \beta\times a_g$

Given the inelastic top story drift ratio the design spectral acceleration can be directly found from Equation 4.16. It should be mentioned that an upper bound on the inelastic top story drift ratio is provided by the design specifications. In general the rules are given for interstory drift but top story drift is the summation of interstory drifts. For example if the interstory drift limit is set at 0.02 then it is ensured that the top story drift cannot exceed this amount too.

As mentioned before, the inelastic top story drift ratio can be extracted from the designs conducted in Chapter 3. A careful examination of the data reveals that this ratio depends on the level of seismicity, importance class, and type of steel used. The ξ factor defined in Chapter 3 was found to represent the variation of inelastic top story drift ratio.

Inelastic top story drift ratios are plotted in Figures 4.3 through 4.8. These plots reveal that the inelastic top story drift ratio increases as the level of seismicity represented as ξ increases. In general the value reaches to an upper bound which is close to the bound set forth by Eurocode 8. In order to represent the data in a simple way the following relationship is proposed herein:

$$\varsigma_I = a \times \xi \leq b \tag{4.17}$$

Where a: slope of the curve fit line in the ascending region, b: upper bound on the inelastic top story drift ratio. The curve fits are also shown in Figures 4.3 through 4.8. The values of the parameters a and b are given in Tables 4.1 and 4.2 for S235 and S355 steels, respectively.

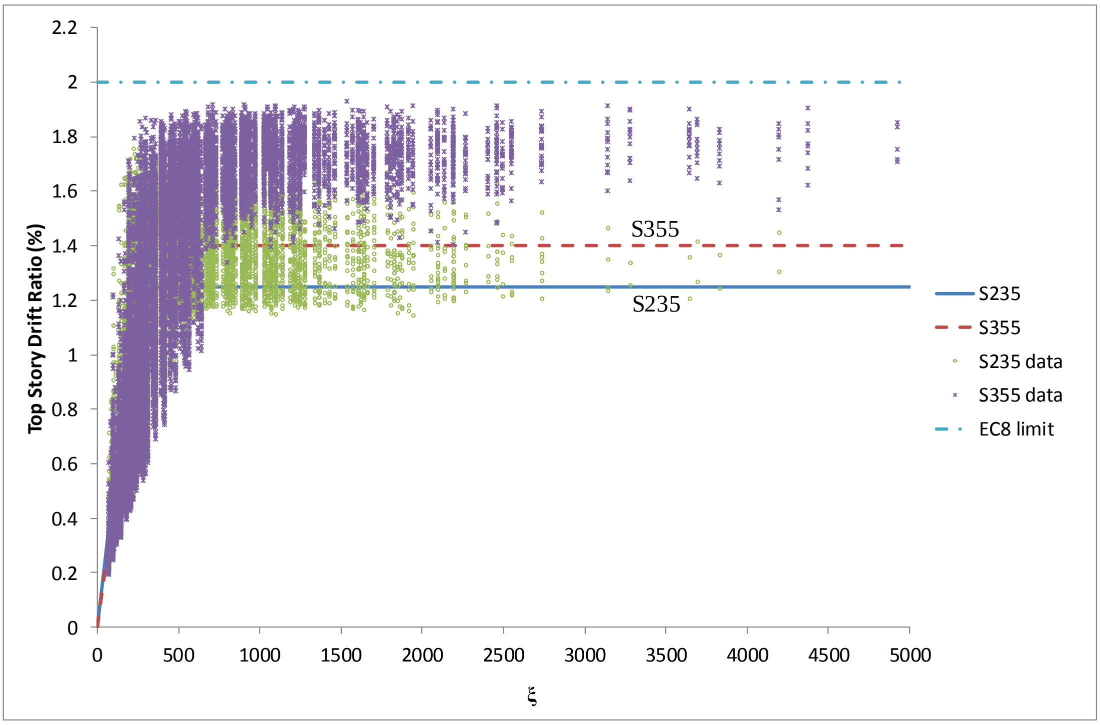


Figure 4.3: Inelastic Top Story Drift Ratios for IC II with Overstrength Rule

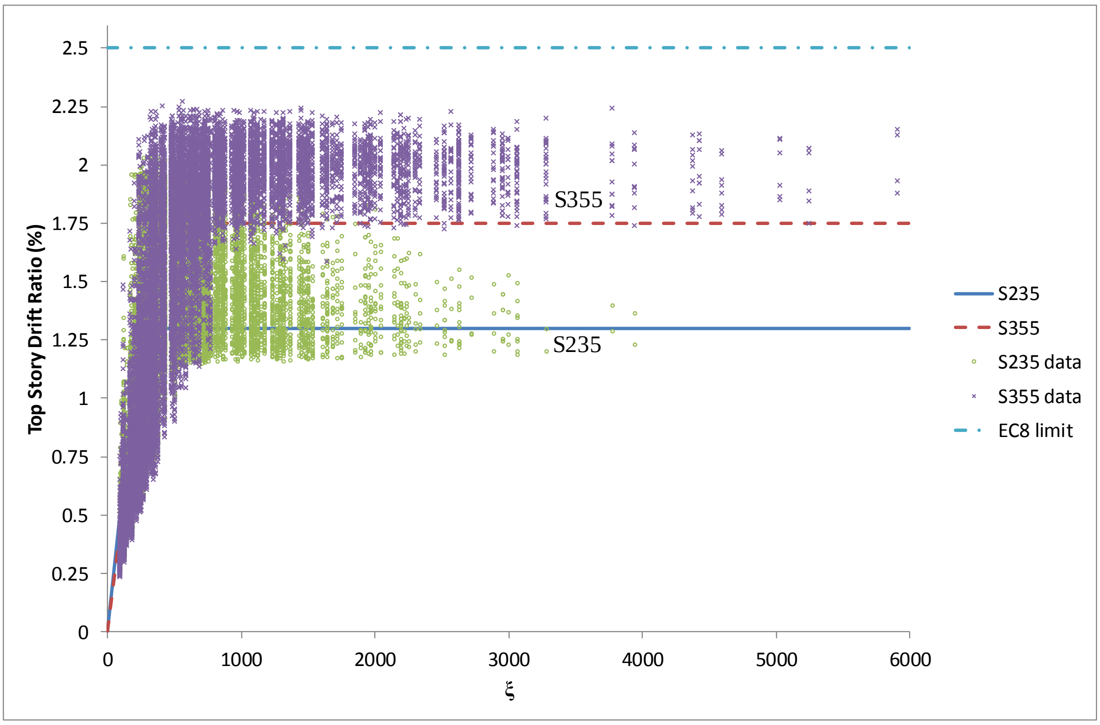


Figure 4.4: Inelastic Top Story Drift Ratios for IC III with Overstrength Rule

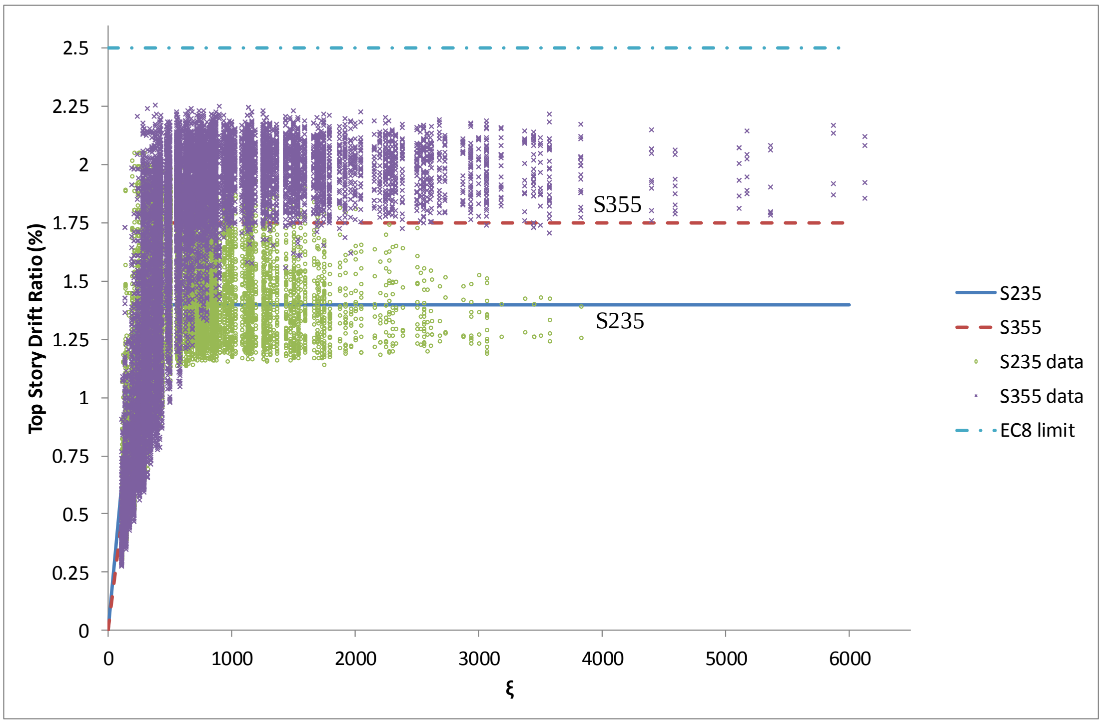


Figure 4.5: Inelastic Top Story Drift Ratios for IC IV with Overstrength Rule

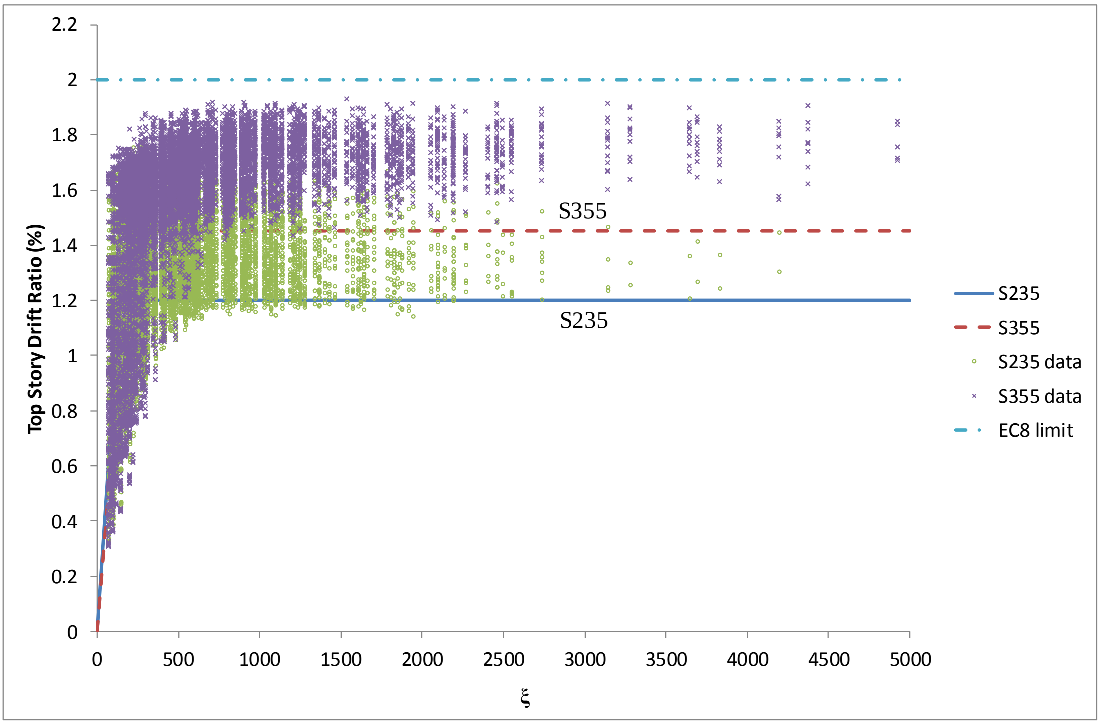


Figure 4.6: Inelastic Top Story Drift Ratios for IC II without Overstrength Rule

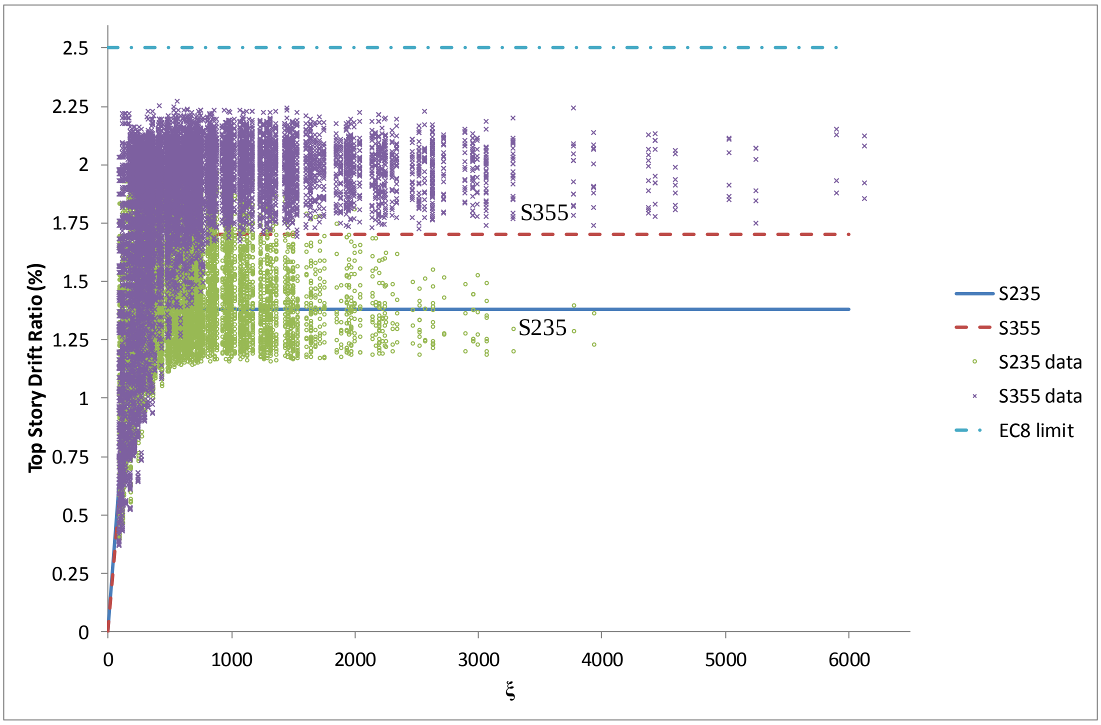


Figure 4.7: Inelastic Top Story Drift Ratios for IC III without Overstrength Rule

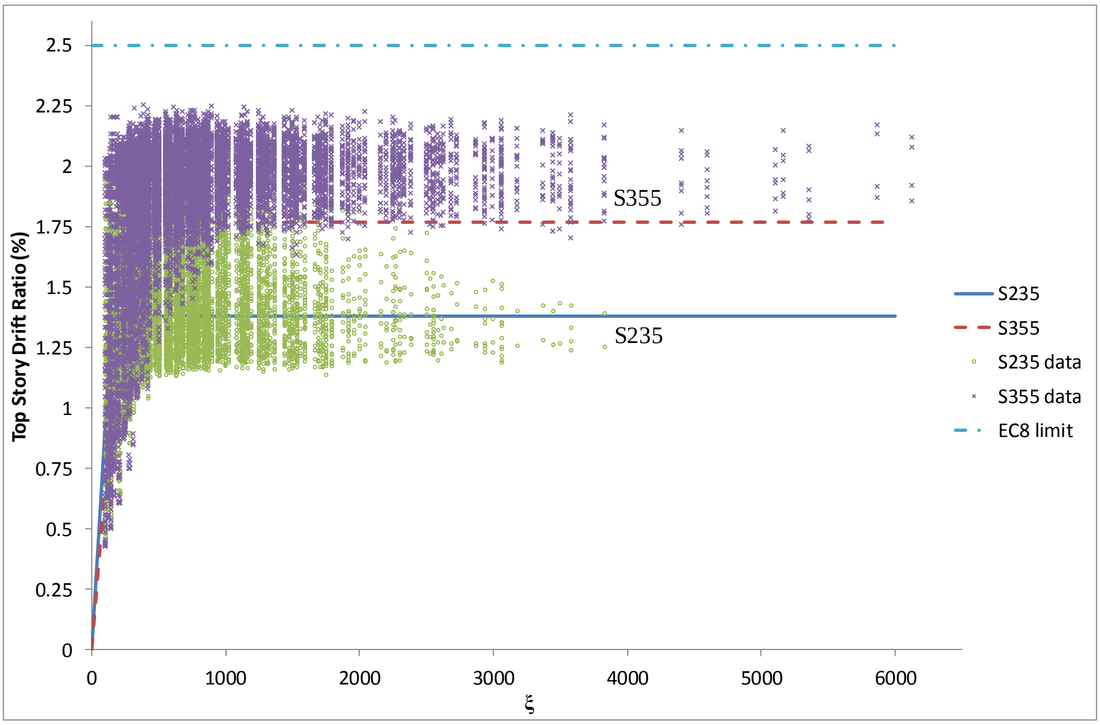


Figure 4.8: Inelastic Top Story Drift Ratios for IC IV without Overstrength Rule

Table 4-1: Statistics of Top Story Drift Ratios for S235

	With Overstrength Rule			Without Overstrength Rule		
Importance Class	IC II	IC III	IC IV	IC II	IC III	IC IV
a	5.21×10^{-5}	5.42×10^{-5}	5.83×10^{-5}	8.57×10^{-5}	6.9×10^{-5}	9.2×10^{-5}
Cut-off	240	240	240	140	200	150
b	0.0125	0.013	0.014	0.012	0.0138	0.0138

Table 4-2: Statistics of Top Story Drift Ratios for S355

	With Overstrength Rule			Without Overstrength Rule		
Importance Class	IC II	IC III	IC IV	IC II	IC III	IC IV
a	4.38×10^{-5}	4.61×10^{-5}	4.49×10^{-5}	7.25×10^{-5}	6.07×10^{-5}	6.81×10^{-5}
Cut-off	320	380	390	200	280	260
b	0.014	0.0175	0.0175	0.0145	0.017	0.0177

In general, the upper bound values (*b* values) tend to increase as the importance class changes from II to IV. In addition, the b values are greater for S355 cases when compared with S235 cases. This indicates that when using higher strength steel smaller members are used and structures become more flexible. When the overstrength rule is omitted its influence on the b values is quite low. However, the slope of the ascending branch changes significantly depending on whether overstrength rule is adopted or not. The following sections present the verification of this proposed method.

4.2 Verification Using Actual Inelastic Top Story Drift Values

The proposed method is first applied using the top story drift values obtained from analysis results. In other words, the top story drift values obtained after the final design is directly input into Equation 4.16 to obtain an estimate of the design spectral acceleration. A comparison of the actual spectral accelerations and estimated ones are given in Figure 4.9 for Importance Class II and including the overstrength provision. The results reveal that the method has a potential to be used to estimate the design spectral accelerations of braced steel frames. The average, standard deviation, maximum, and minimum of the Estimated/Actual ratios are 0.98, 0.03, 1.08, and 0.84, respectively.

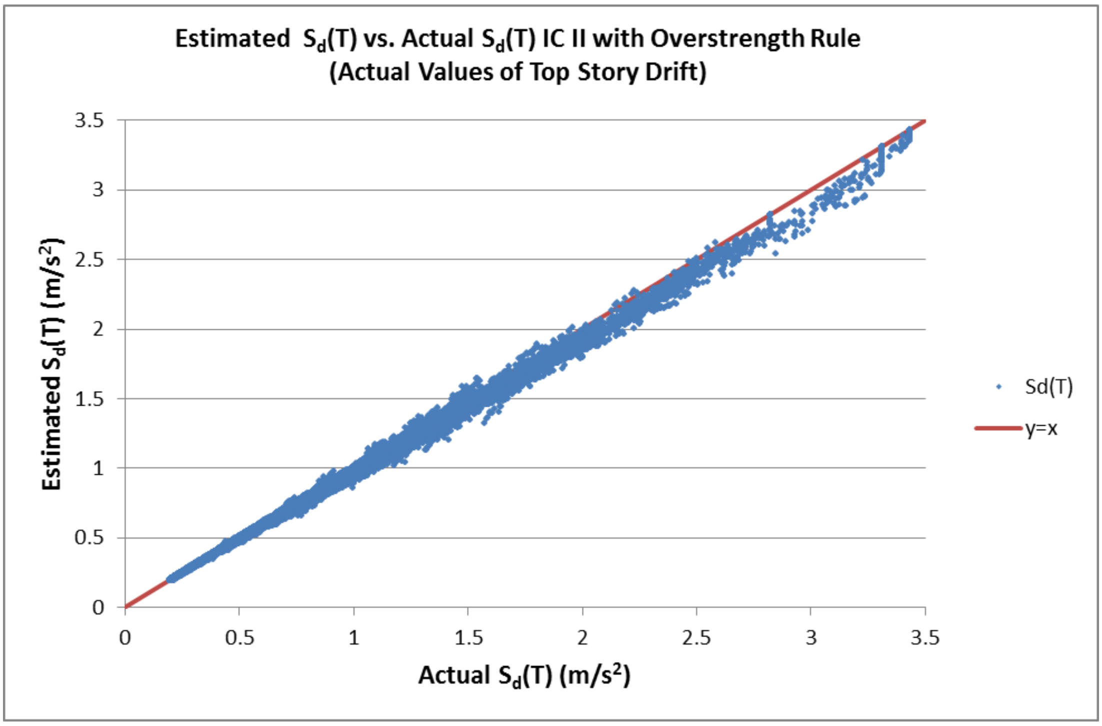


Figure 4.9: Estimated vs. Actual S_d(T) IC II with Overstrength Rule (Actual Values of Top Story Drift)

4.3 Verification Using Estimated Inelastic Top Story Drift Values

As a last step the method was verified by using the inelastic top story drifts estimated using Equation 4.17. The results are presented in Figures 4.10 through 4.15. The statistical measures are also given in Table 4.3.

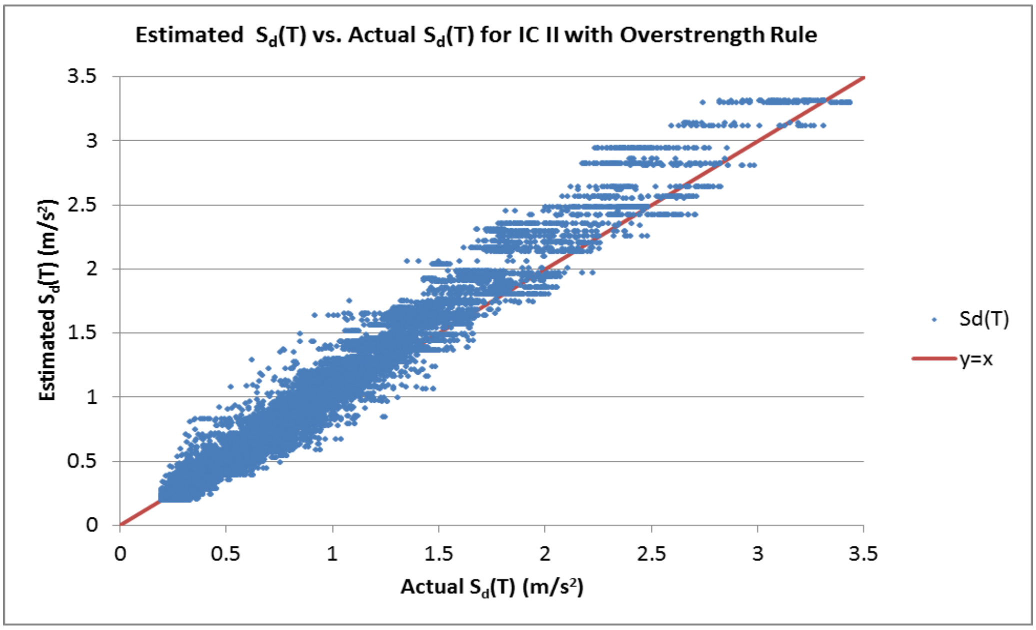


Figure 4.10: Estimated vs. Actual S_d(T) IC II with Overstrength Rule

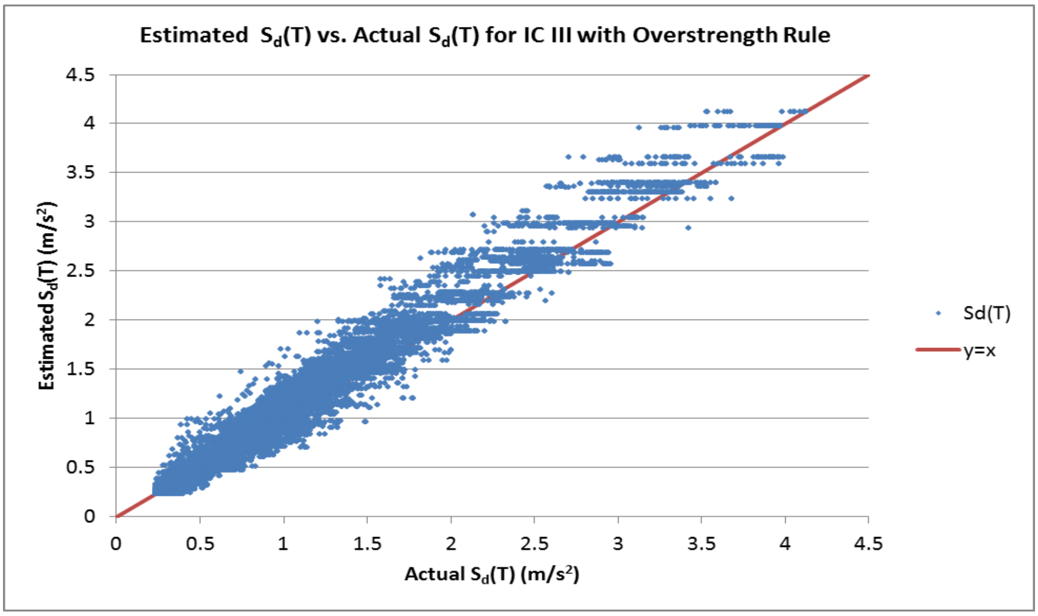


Figure 4.11: Estimated vs. Actual S_d(T) IC III with Overstrength Rule

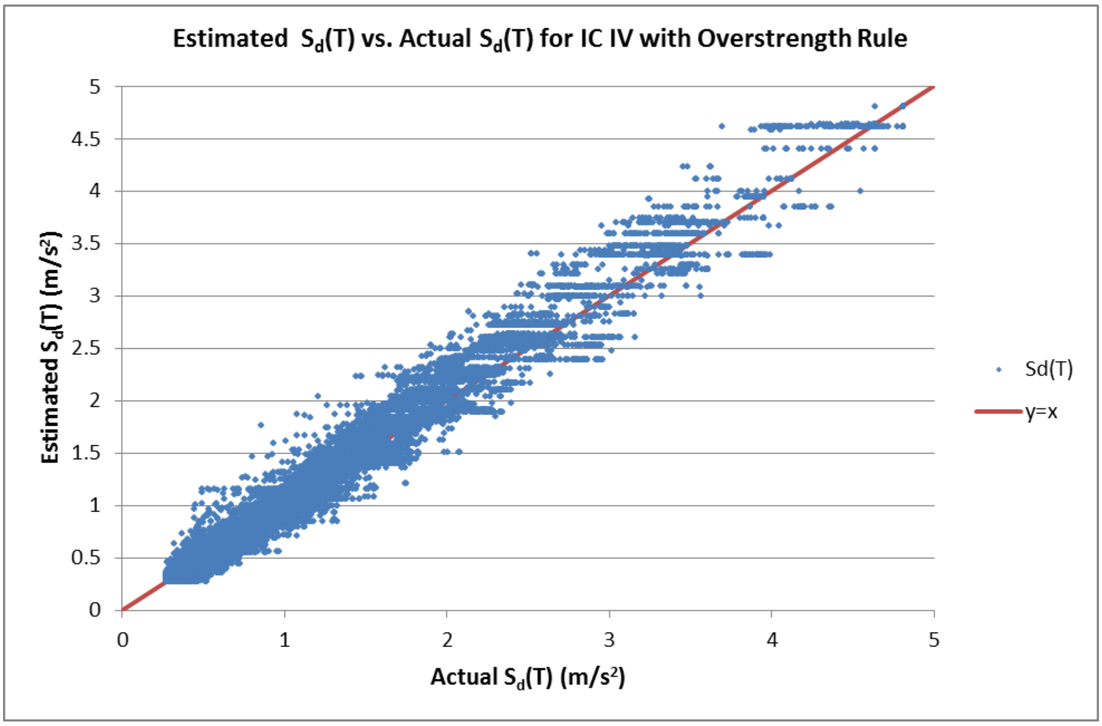


Figure 4.12: Estimated vs. Actual $S_d(T)$ IC IV with Overstrength Rule

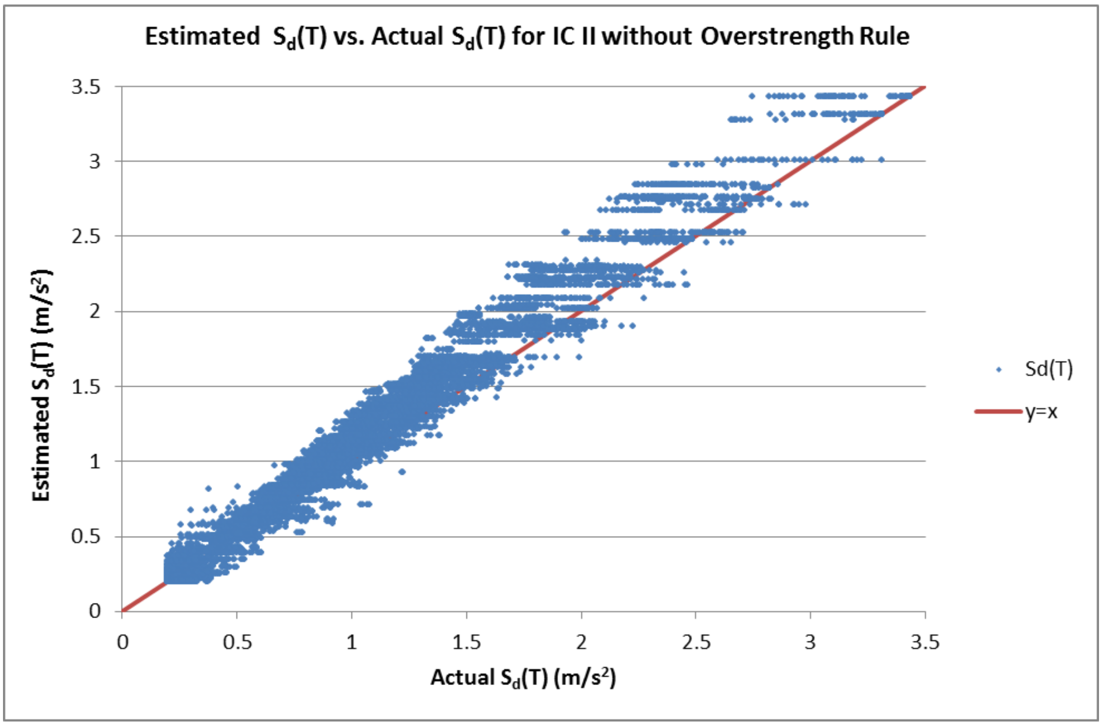


Figure 4.13: Estimated vs. Actual $S_d(T)$ IC II without Overstrength Rule

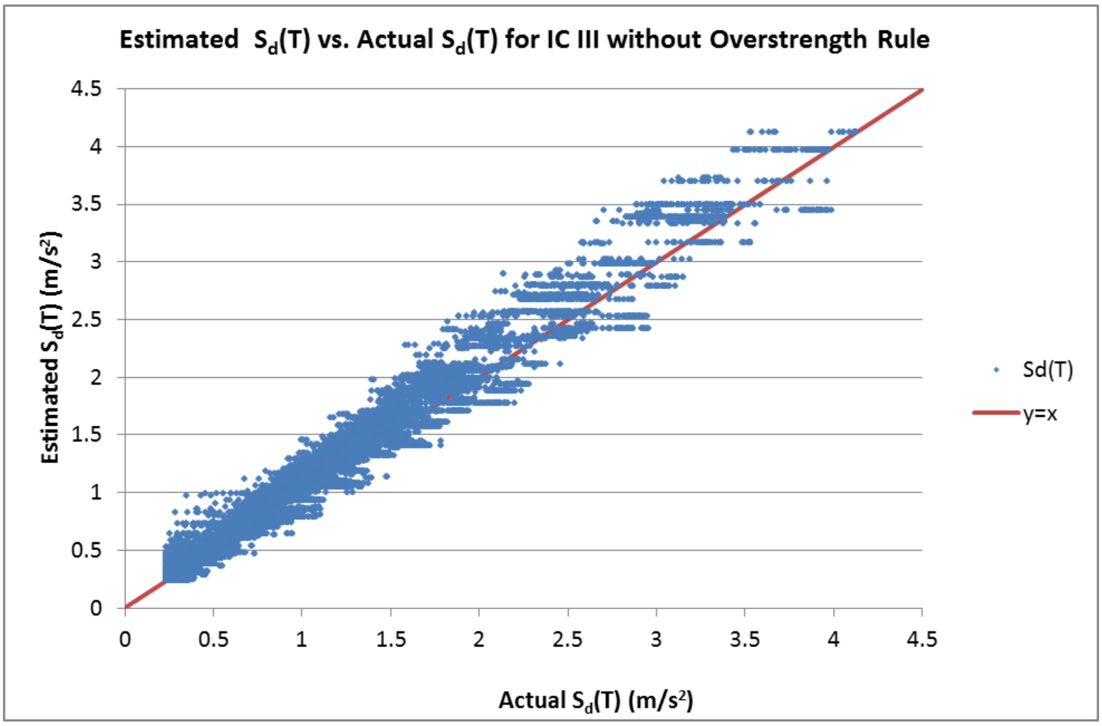


Figure 4.14: Estimated vs. Actual $S_d(T)$ IC III without Overstrength Rule

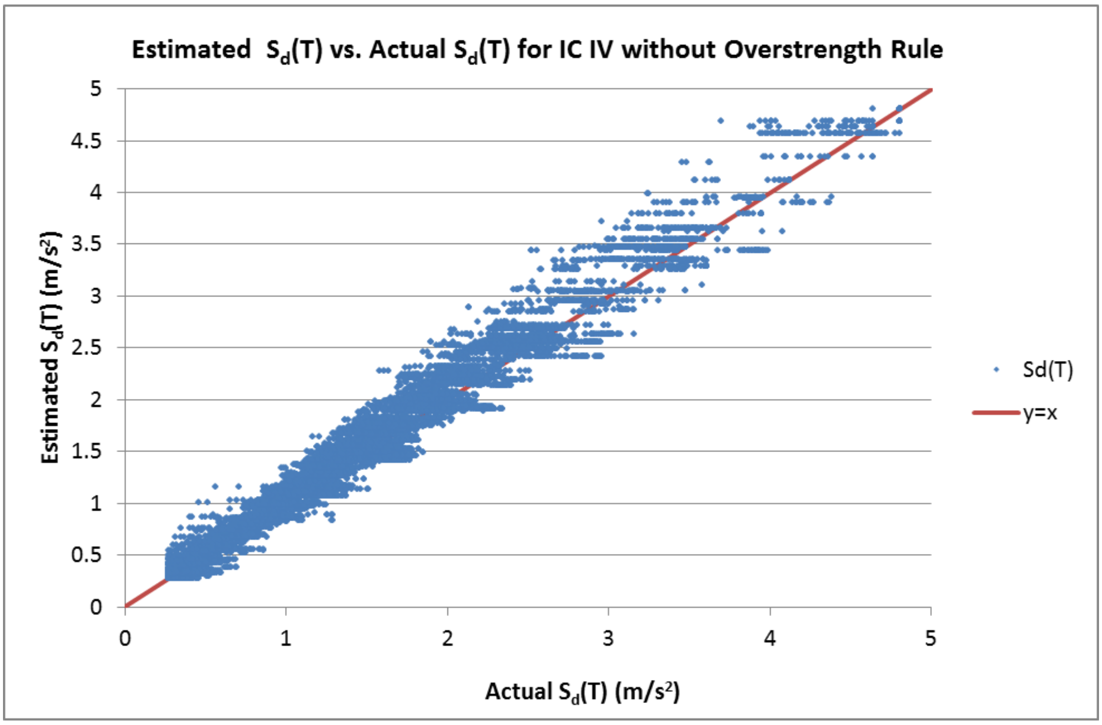


Figure 4.15: Estimated vs. Actual $S_d(T)$ IC IV without Overstrength Rule

Table 4-3: Statistics of Verification Data

	With Overstrength Rule			Without Overstrength Rule		
Estimated $S_d(T)$ / Actual $S_d(T)$	IC II	IC III	IC IV	IC II	IC III	IC IV
Mean	1.06	1.01	1.00	1.05	1.03	1.02
Standard Deviation	0.13	0.12	0.11	0.10	0.11	0.09
Max.	2.46	2.40	2.34	2.28	2.92	2.19
Min.	0.56	0.55	0.53	0.55	0.64	0.55

Based on the statistical values it is evident that the method estimates the design spectral accelerations with reasonable accuracy. In general the averages of the ratios are close to unity. The standard deviations reduce significantly when compared with the results presented in Chapter 3.

CHAPTER 5

CONCLUSIONS

A two-phase research study has been conducted to investigate the fundamental natural periods of braced steel frames designed to Eurocode 8. In the first phase typical office buildings were designed according to Eurocode 8 and two types of designs were conducted. In the first type the initial period is found from the lower bound expression given in Eurocode 8 and the final period is obtained after finalizing the design. The second type is similar to the first one but an updating of periods was conducted so that the assumed and final periods converge in the last design step. These designs are called non-iterative and iterative. In addition, the overstrength rule was included and omitted to results in designs with different overstrength provisions. In general, regardless of the type of design conducted the final periods are longer than the periods obtained by using the lower bound expression given in Eurocode 8. Therefore, it can be concluded that the lower bound expression given in Eurocode 8 can be safely used. The lower bound expression developed by Tremblay (2005) is also acceptable. However, there are some cases that result in shorter periods compared with the estimates provided by Tremblay's expression.

In the second phase a simple expression was developed to estimate the design base acceleration for braced frames designed to Eurocode 8. This method is based on some underlying assumptions and requires the inelastic top story drift a priori. The inelastic top story drift values were obtained from the structures designed in this study using the iterative method. These drifts were represented by simple expressions using curve fitting techniques. When compared with the base accelerations of the designed frames the developed expression provides estimates with acceptable accuracy. It can be concluded that the technique developed in this study can be used in the design of braced frames and provides a fairly good first order estimate of the design base acceleration. It should be mentioned that some further iterations may be needed to come up with the final design. However, it is considered that the present method reduces the amount of iterations required to reach to the final design.

The study was limited to regular frames with regular floor plans. Future research should consider natural periods of irregular frames.

REFERENCES

Eurocode 3: Design of Steel Structures-Part 1-1: General Rules and Rules for Buildings. (2003). Brussels: European Committee for Standardization.

Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. (2004). Brussels: European Committee for Standardization.

Eurocode 1: Actions on structures - Part 1-1: General actions -Densities, self-weight, imposed loads for buildings. (2001). Brussels: European Committee for Standardization.

Eurocode-Basis of Structural Design. (2002). Brussels: European Committee for Standardization.

AISC, 2005. Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05, American Institute of Steel Construction, Chicago, Illinois.

The ASCE Standard 7-10- Minimum Design Loads for Buildings and Other Structures. (2010). American Society of Civil Engineers.

National Building Code of Canada 2005 – Volume 1, Twelfth Edition, Canadian Commission on Building and Fire Codes National Research Council of Canada.

Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik. (2007). Ankara: TMMOB İnşaat Mühendisleri Odası.

Tremblay, R. (2005). Fundamental periods of vibration of braced steel frames for seismic design. *Earthquake Spectra*; **21**(3): 833-860.

Housner, G.W., Brady, A.G. (1963). Natural periods of vibrations of buildings. *Journal of the Engineering Mechanics Division*; **89**(4): 31–65

Goel, R.K., Chopra, A.K. (1998). Period formulas for concrete shear wall buildings. *Journal of Structural Engineering*; **124**(4): 426–433.

Stafford Smith B., Crowe, E. (1986). Estimating periods of vibration of tall buildings. *Journal of Structural Engineering*; **112**(5): 1005–1019.

Goel, R.K., Chopra, A.K. (1997). Vibration properties of buildings determined from recorded earthquake motions. *Report No. EERC-97/14,* Earthquake Engineering Research Center, University of California, Berkeley, Richmond, CA.

Goel, R.K., Chopra, A.K. (1997). Period formulas for moment-resisting frame buildings. *Journal of Structural Engineering*; **123**(11): 1454–1461.

Zalka, K.A. (2001). A simplified method for calculation of the natural frequencies of wall-frame buildings. *Engineering Structures*; **23**: 1544–1555.

Heidebrecht, A.C., Stafford Smith, B. (1973). Approximate analysis of tall wall-frame structures. *Journal of the Structural Division*; ASCE **99**(2): 199–221.

Chrysanthakopoulos, C., Bazeos, N., Beskos, D.E. (2006). Approximate formulae for natural periods of plane steel frames. *Journal of Constructional Steel Research*; **62**(6): 592–604.

Crowley, H., Pinho, R. (2010). Revisiting Eurocode 8 formulae for periods of vibration and their employment in linear seismic analysis. *Earthquake Engineering and Structural Dynamics*; **39**(2): 223–235.

Oh-Sung Kwon, Eung Soo Kim. (2010). Evaluation of building period formulas for seismic design. *Earthquake Engineering and Structural Dynamics*; **39**(14): 1569-1583.

Canney, N. (2004). Performance of concentrically braced frames under cyclic loading. *Proceedings of the 2004 Earthquake Engineering Symposium for Young Researchers*, Charleston, South Carolina.

Uriz, P., Mahin, S.A. (2004). Seismic performance assessment of concentrically braced frames. *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, paper no. 1639.

The SEAOC Blue Book: Seismic Design Recommendations, Structural Engineers Association of California, Sacramento, CA; 2008.

Fardis, M., Carvalho, E., Elnashai, A., Faccioli, E., Pinto, P., Plumier A. (2005). Designers' Guide to EN 1998-1 and 1998-5 Eurocode 8: Design of Structures for Earthquake Resistance; General Rules, Seismic Actions, Design rules for buildings, foundations and retaining structures., First Edition, Thomas Telford Publishing Ltd, 1 Heron Quay, London.

Elghazouli, A.Y. (2009). Seismic Design of Buildings to Eurocode 8., First Edition, Spon Press, 2 Park Square, Abingdon, Oxon, UK.

APPENDIX A

ILLUSTRATIONS

This section includes four examples of X-bracing (drift governed design and strength governed design), Split-X bracing (drift governed design and strength governed design) which were calculated by the developed MATLAB code.

- The first example is as follows:

The first example is a X-bracing system which has 8 stories and two braces in plan as shown in the Chapter 2/Figure 2.2. Braced bay width is 6 m. Seismic zone and soil class are 2 and A, respectively. Seismic zone 2 result in 0.3g for a_g. Story height is 4 m which leads to 32 m of total height for the structure. Brace lengths can be calculated as 7.2 m. For story mass calculations, load intensity is obtained according to Eqn. 2.9 as 4.7 kN/m^2 per story.

$$\text{m}=\sum G_{k,j}\text{"+"}\sum \psi_{E,i}.Q_{k,i} = 4.4+0.15\times 2 = 4.7\ \text{kN/m}^2$$

Then, story mass is equal to 158.1 tons per braced bay.

$$storymass = 4.7\times 22\times 30/9.81/2 = 158\ \text{tons}$$

Design starts with a 0.673 sec initial period value.

$$T_1 = 0.05\times H^{3/4} = 0.05\times 32^{3/4} = 0.673\ \text{sec}$$

Gravity load contribution is calculated according to Eqn. 2.25. $G+0.3Q=4.4+0.3\text{ x }2=5\text{ kN/m}^2$ and columns in the braced bay has 35 m^2 tributary area. So, it is equal to 35x5=175 kN per story for columns in the braced bay. Base shear is determined as 1528.6 kN by conducting the following calculations.

$$T_C = 0.4,\ T_B = 0.15,\ T_1 = 0.673 \qquad T_C < T_1 < T_D$$
$$Eqn\ 2.3\ \text{gives} \quad S_d(T) = 0.3 \times g \times 1 \times \frac{2.5}{4} \times \frac{0.4}{0.673} = 1.093\text{ m/s}^2$$
$$T_1 < 2T_C \quad \text{building has more than two stories } \lambda\text{=0.85}$$
$$F_b = 1.093 \times 158 \times 8 \times 0.85 \times 1.3 = 1526.6\text{ kN}$$

Torsion effects are included through the 1.3 factor.

Lateral forces resulting from distribution of base shear, right and left column forces can be seen in Table A-1 and in Figure A.1. Tension is positive (+), compression is negative (-).

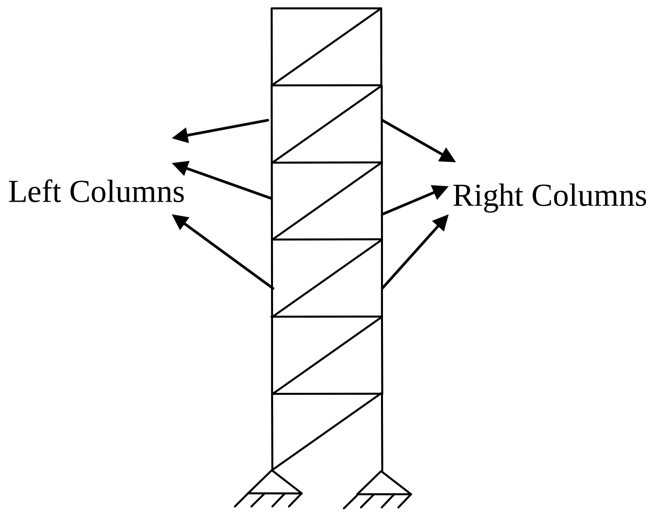


Figure A.1: Description of Left and Right Columns

Table A-1: Forces Acting on Stories and Columns Forces

Story Number	Forces Acting on Stories (kN)	Right Column Forces (kN)	Left Column Forces (kN)
1	42.5	-7174.7	3355.6
2	84.9	-5980.6	2539.9
3	127.4	-4814.9	1780.7
4	169.8	-3705.7	1106.5
5	212.3	-2681.5	545.5
6	254.8	-1770.5	126.1
7	297.2	-1001.1	-123.5
8	339.7	-401.5	-175

Then brace forces are calculated and braces are designed for strength (Eqn. 2.14, 2.16) and slenderness (Eqn. 2.22) rules. The resulting brace sections are showed in Table A.2.

Table A-2: Brace Forces and First Design for Braces

Story Number	Brace Forces (kN)	Brace section Areas for Strength and Slenderness (mm^2)	Brace Sections	Omega (Ω) Values	Unit Weight (kg/m)
1	1837	5260	RHS x 140 x160 x 10	1.0164	41.3
2	1786	5120	RHS x 150 x 200 x 8	1.0176	40.2
3	1684	4840	RHS x 150 x 350 x 5	1.0203	38.0
4	1531	4320	RHS x 125 x 175 x 8	1.0017	33.9
5	1327	3790	CHS x 159 x 8	1.0140	29.8
6	1072	3020	CHS x 159 x 6.3	1.0004	23.7
7	766	2180	CHS x 177.8 x 4	1.0110	17.1
8	408	1470	CHS x 159 x 3	1.2782	11.5

Table A-2: Brace Forces and First Design for Braces-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	1867.3	562.2	1481	1.82
2	1817.6	689.2	1816	1.62
3	1718.2	772.0	2034	1.49
4	1533.6	398.4	1050	1.96
5	1345.5	411.8	1085	1.81
6	1072.1	334.9	882	1.79
7	773.9	313.2	825	1.57
8	521.9	169.8	447	1.75

As shown in Table A-2 the overstrength of the braces does not satisfy the rule presented in EC8, therefore, the braces sizes of the bottom stories were adjusted to satisfy this rule. The resulting sections are given in Table A-3.

In design of columns, compression effects can cause buckling which is more unsafe than a yielding arising from tension effects. So, compression forces govern in design of columns.

Table A-3: Brace Sections after Overstrength Check

Story Number	Brace Section Areas for Omega Rule (mm^2)	Brace Sections for Omega Rule	Omega (Ω) Values	Unit Weight (kg/m)
1	5383	HE 200 A	1.0402	42.3
2	5260	RHS x 140 x160 x 10	1.0455	41.3
3	5120	RHS x 150 x 200 x 8	1.0793	40.2
4	4560	RHS x 150 x 250 x 6	1.0574	35.8
5	3840	RHS x 150 x 250 x 5	1.0274	30.1
6	3210	CHS x 168.3 x 6.3	1.0633	25.2
7	2290	RHS x 140 x160 x 4	1.0620	18.0
8	1470	CHS x 159 x 3	1.2782	11.5

Table A-3: Brace Sections after Overstrength Check-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	1911.0	507.1	1336	1.94
2	1867.3	562.2	1481	1.82
3	1817.6	689.2	1816	1.62
4	1618.8	671.2	1768	1.55
5	1363.2	572.4	1508	1.54
6	1139.6	399.9	1053	1.69
7	812.9	275.4	726	1.72
8	521.9	169.8	447	1.75

New omega values of braces are shown in Table A-4. The ratio of Ω_{max} to Ω_{min} is 1.245. The minimum value of 1.027 is used in the design of columns.

Table A-4: Preliminary Column Design

Story Number	Column Section Areas for Min. Omega (mm^2)	Column Sections for Min. Omega	Unit Weight (kg/m)	Moment of Inertia (cm^4)
1	31900	HE 360 M	250.0	19520
2	31900	HE 360 M	250.0	19520
3	22800	HD 360 x 179	179.0	20680
4	22800	HD 360 x 179	179.0	20680
5	13400	HE 340 A	105.0	7440
6	13400	HE 340 A	105.0	7440
7	6400	HE 220 A	50.5	1960
8	6400	HE 220 A	50.5	1960

Table A-4: Preliminary Column Design-(continued)

Story Number	Column N_{CR} (kN)	Imperfection Factor (α)	Reduction Factor (χ)	Φ
1	24082	0.34	0.7918	0.8175
2	24082	0.34	0.7918	0.8175
3	25513	0.49	0.8067	0.7479
4	25513	0.49	0.8067	0.7479
5	9174	0.49	0.7130	0.8854
6	9174	0.49	0.7130	0.8854
7	2412	0.49	0.5559	1.1626
8	2412	0.49	0.5559	1.1626

After all members are designed, displacements at all stories are determined and they are given in Table A-5:

Table A-5: Displacements of the Structure

Story Number	Story Displacements (m)	Interstory Drifts (m)	Interstory Drift Ratio ($\delta R/h_s$)
1	0.0178	0.0178	1.78%
2	0.0394	0.0216	2.16%
3	0.0644	0.0250	2.50%
4	0.0930	0.0285	2.85%
5	0.1253	0.0323	3.23%
6	0.1593	0.0341	3.41%
7	0.1957	0.0363	3.63%
8	0.2301	0.0344	3.44%

The resulting structure has a period of 1.99 sec.

Interstory drift ratios exceed the limits stated in Eqn. 2.27, so they should be decreased to a maximum value of 0.02 as shown in Table A-6.

All member cross-sections are multiplied with a coefficient 3.63/2=1.8154 to satisfy displacement provisions.

It should be checked again that brace overstrengths are close each other with respect to homogenous behavior. $\frac{\Omega_{max}}{\Omega_{min}} = \frac{2.3565}{1.9331} = 1.22 < 1.25$ OK.

Table A-6: Recalculation of Omega (Ω) Values for Modified Design and Final Brace Design

Story Number	New Omega Values (Ω)	Brace Section Areas (mm^2)	Brace Sections	Unit Weight (kg/m)
1	2.0483	10600	HE 240 B	83.2
2	1.9331	9726	HE 280 A	76.4
3	2.0503	9726	HE 280 A	76.4
4	2.0132	8682	HE 260 A	68.2
5	1.9425	7260	RHS x 150 x 250 x 10	57.0
6	1.9611	5920	RHS x 150 x 250 x 8	46.5
7	1.9803	4270	CHS x 177.8 x 8	33.5
8	2.3565	2710	CHS x 177.8 x 5	21.3

Table A-6: Recalculation of Omega (Ω) Values for Modified Design and Final Brace Design-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	3763.0	1489.2	3923	1.5896
2	3452.7	1808.0	4763	1.3819
3	3452.7	1808.0	4763	1.3819
4	3082.1	1392.4	3668	1.4878
5	2577.3	999.9	2634	1.6054
6	2101.6	842.4	2219	1.5795
7	1515.9	585.1	1541	1.6096
8	962.1	384.9	1014	1.5809

In the end, final column sizes, displacements and fundamental period of the structure can be calculated as follows in Table A-7 and A-8:

Table A-7: Final Column Sizes and Properties

Story Number	Column Section Areas (mm^2)	Column Sections	Unit Weight (kg/m)	Moment of Inertia (cm^4)	Radius of Gyration (mm)
1	58950	HD 400 x 463	463.0	67040	10.66
2	58950	HD 400 x 463	463.0	67040	10.66
3	44200	HD 400 x 347	347.0	48090	10.43
4	44200	HD 400 x 347	347.0	48090	10.43
5	25030	HD 360 x 196	196.0	22860	9.56
6	25030	HD 360 x 196	196.0	22860	9.56
7	11800	HE 260 B	93.0	5140	6.58
8	11800	HE 260 B	93.0	5140	6.58

Table A-7: Final Column Sizes and Properties-(continued)

Story Number	Column N_{CR} (kN)	Imperfection Factor (α)	Reduction Factor (χ)	Φ	Column Slenderness (λ)
1	82707	0.49	0.8413	0.7008	0.503
2	82707	0.49	0.8413	0.7008	0.503
3	59329	0.49	0.8350	0.7092	0.514
4	59329	0.49	0.8350	0.7092	0.514
5	28202	0.49	0.8081	0.7461	0.561
6	28202	0.49	0.8081	0.7461	0.561
7	6335	0.49	0.6530	0.9823	0.814
8	6335	0.49	0.6530	0.9823	0.814

Table A-8: Final Column Forces and Capacities

Story Number	Column Forces (kN)	Column Capacities (kN)	Column Overstrengths
1	-7174.7	17606	2.45
2	-5980.6	17606	2.94
3	-4814.9	13102	2.72
4	-3705.7	13102	3.54
5	-2681.5	7180	2.68
6	-1770.5	7180	4.06
7	-1001.1	2745	2.74
8	-401.5	2745	6.84

Table A-9: Final Displacement Values

Story Number	Displacements (m)	Interstory Drifts (m)	Interstory Drift Ratio ($\delta R/h_s$)
1	0.0092	0.0092	0.92%
2	0.0209	0.0118	1.18%
3	0.0344	0.0134	1.34%
4	0.0497	0.0153	1.53%
5	0.0672	0.0175	1.75%
6	0.0858	0.0187	1.87%
7	0.1056	0.0198	1.98%
8	0.1245	0.0189	1.89%

Fundamental period of the structure is equal to 1.45 sec in conclusion.

If the developed iterative method is used for this example, results would be as follows:

Design starts with a 1.7 sec initial period value. Base shear is determined as 967.19 kN by conducting the following calculations.

$T_C = 0.4,\ T_B = 0.15,\ T_1 = 1.7 \qquad T_C < T_1 < T_D$

Eqn 2.3 gives $\quad S_d(T) = 0.3 \times g \times 1 \times \dfrac{2.5}{4} \times \dfrac{0.4}{1.7} = 0.433 \text{ m/s}^2$

0.433 must be bigger than $0.2a_g = 0.2 \times 0.3 \times 9.81 = 0.5886$

$T_1 > 2T_C \qquad \lambda = 1$

$F_b = 0.5886 \times 158 \times 8 \times 1 \times 1.3 = 967.19$ kN

Table A-10: Forces Acting on Stories and Columns Forces

Story Number	Forces Acting on Stories (kN)	Right Columns Forces (kN)	Left Columns Forces (kN)
1	26.9	-5056.2	1611
2	53.8	-4236	1158.7
3	80.7	-3433.7	742.3
4	107.5	-2667.3	379.6
5	134.4	-1954.6	88.6
6	161.3	-1313.6	-112.8
7	188.2	-762.2	-206.6
8	215.1	-318.4	-175

Final brace and column design results are indicated in the following tables.

Table A-11: Final Brace Design

Story Number	Brace Forces (kN)	Omega Values (Ω)	Brace Section Areas (mm^2)	Brace Sections	Unit Weight (kg/m)
1	1163.2	2.2157	7260	RHS x 150 x 250 x 10	57.0
2	1130.9	2.2790	7260	RHS x 150 x 250 x 10	57.0
3	1066.2	2.1874	6570	CHS x 219.1 x 10	51.6
4	969.3	2.1681	5920	RHS x 150 x 250 x 8	46.5
5	840.1	2.1636	5120	RHS x 150 x 200 x 8	40.2
6	678.5	2.1503	4110	RHS x 150 x 200 x 6.3	32.3
7	484.7	2.2121	3020	CHS x 159 x 6.3	23.7
8	258.5	2.6781	1950	CHS x 159 x 4	15.3

Table A-11: Final Brace Design-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	2577.3	999.9	2634	1.61
2	2577.3	999.9	2634	1.61
3	2332.4	1366.0	3598	1.31
4	2101.6	842.4	2219	1.58
5	1817.6	689.2	1816	1.62
6	1459.1	569.1	1499	1.60
7	1072.1	335.0	882	1.79
8	692.3	222.2	585	1.77

Table A-12: Final Column Sizes and Properties

Story Number	Column Section Areas (mm^2)	Column Sections	Unit Weight (kg/m)	Moment of Inertia(cm^4)	Radius of Gyration (mm)
1	48710	HD 400 x 382	382.0	53620	10.49
2	48710	HD 400 x 382	382.0	53620	10.49
3	31880	HE 360 M	250.0	19520	7.43
4	31880	HE 360 M	250.0	19520	7.43
5	18790	HD 360 x 147	147.0	16720	9.43
6	18790	HD 360 x 147	147.0	16720	9.43
7	8680	HE 260 A	68.2	3668	6.50
8	8680	HE 260 A	68.2	3668	6.50

Table A-12: Final Column Sizes and Properties-(continued)

Story Number	Column N_{CR} (kN)	Imperfection Factor (α)	Reduction Factor (χ)	Φ	Column Slenderness (λ)
1	66151	0.49	0.8367	0.7070	0.511
2	66151	0.49	0.8367	0.7070	0.511
3	24082	0.34	0.7918	0.8175	0.686
4	24082	0.34	0.7918	0.8175	0.686
5	20627	0.49	0.8038	0.7520	0.569
6	20627	0.49	0.8038	0.7520	0.569
7	4525	0.49	0.6463	0.9936	0.825
8	4525	0.49	0.6463	0.9936	0.825

Table A-13: Final Column Forces and Capacities

Story Number	Column Forces (kN)	Column Capacities (kN)	Column Overstrengths
1	-5056.2	14468	2.86
2	-4236	14468	3.42
3	-3433.7	8961	2.61
4	-2667.3	8961	3.36
5	-1954.6	5362	2.74
6	-1313.6	5362	4.08
7	-762.2	1992	2.61
8	-318.4	1992	6.26

Fundamental period of the structure is equal to 1.7 sec in final step.

Non-iterative method gives a total weight of 5321 kN/m while iterative method gives a total weight of 4036 kN/m.

- The second example is as follows:

The second example is a split-X bracing system which has 8 stories and four braces in plan. Braced bay width is 14 m. Seismic zone and soil class are 4 and E, respectively. Seismic zone 4 result in 0.1g for a_g. Story height is 4 m which leads to 32 m of total height for the structure. Brace lengths can be calculated as 14.56 m. Plan type is chosen as in the Fig 2.3. For story mass calculations, load intensity is obtained according to Eqn. 2.9. It is equal to 4.7 kN/m^2 per story. Then, story mass is equal to 79.05 tons. Design starts with a 0.6727 sec initial period value. Gravity load contribution is calculated according to Eqn. 2.25 and it is equal to 225 kN per story. Base shear is calculated as 445.8406 kN by using these values.

Lateral forces resulting from distribution of base shear, right and left column forces can be seen in Table A-14.

Table A-14: Forces Acting on Stories and Columns Forces

Story Number	Forces Acting on Stories (kN)	Right Column Forces (kN)	Left Column Forces (kN)
1	12.4	-2521.8	-1332.9
2	24.8	-2293.3	-1104.4
3	37.2	-1820.6	-1112.9
4	49.5	-1585	-877.3
5	61.9	-1147.7	-836.3
6	74.3	-905	-593.6
7	86.7	-531.4	-474.8
8	99.1	-281.6	-225

Then brace forces are calculated and braces are designed for strength (Eqn. 2.14, 2.16) and slenderness (Eqn. 2.22) rules. It is shown in Table A-15.

Table A-15: Brace Forces and First Design for Braces

Story Number	Brace Forces (kN)	Brace Section Areas for Strength and Slenderness (mm^2)	Brace Sections	Omega (Ω) Values	Unit Weight (kg/m)
1	2522	1470	CHS x 159 x 3	1.0163	11.5
2	2293	1470	CHS x 159 x 3	1.0453	11.5
3	1821	1470	CHS x 159 x 3	1.1087	11.5
4	1585	1470	CHS x 159 x 3	1.2195	11.5
5	1148	1470	CHS x 159 x 3	1.4071	11.5
6	905	1470	CHS x 159 x 3	1.7422	11.5
7	531	1470	CHS x 159 x 3	2.4390	11.5
8	282	1470	CHS x 159 x 3	4.5732	11.5

Table A-15: Brace Forces and First Design for Braces-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	521.9	135.9	447.4	1.96
2	521.9	135.9	447.4	1.96
3	521.9	135.9	447.4	1.96
4	521.9	135.9	447.4	1.96
5	521.9	135.9	447.4	1.96
6	521.9	135.9	447.4	1.96
7	521.9	135.9	447.4	1.96
8	521.9	135.9	447.4	1.96

As shown in Table A-15 the overstrength of the braces does not satisfy the rule presented in EC8, therefore, the braces sizes of the bottom stories were adjusted to satisfy this rule. The resulting sections are given in Table A-16.

In design of columns, compression effects can cause buckling which is more unsafe than a yielding arising from tension effects. So, compression forces govern in design of columns.

Table A-16: Brace Sections after Overstrength Check

Story Number	Brace Section Areas for Omega Rule (mm^2)	Brace Sections for Omega Rule	Omega (Ω) Values	Unit Weight (kg/m)
1	5310	CHS x 219.1 x 8	3.6710	41.6
2	5310	CHS x 219.1 x 8	3.7759	41.6
3	5120	RHS x 150 x 200 x 8	3.8614	40.2
4	4560	RHS x 150 x 250 x 6	3.7830	35.8
5	3840	RHS x 150 x 250 x 5	3.6758	30.1
6	3210	CHS x 168.3 x 6.3	3.8043	25.2
7	2290	RHS x 140 x160 x 4	3.7996	18.0
8	1470	CHS x 159 x 3	4.5732	11.5

Table A-16: Brace Sections after Overstrength Check-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	1885.1	898.8	2959.6	1.45
2	1885.1	898.8	2959.6	1.45
3	1817.6	551.3	1815.5	1.82
4	1618.8	537.0	1768.3	1.74
5	1363.2	457.9	1508	1.73
6	1139.6	319.9	1053.4	1.89
7	813.0	220.4	725.6	1.92
8	521.9	135.9	447.4	1.96

New omega values of braces are shown in Table A-16. The ratio of Ω_{max} to Ω_{min} is 1.246. The minimum value of 3.671 is used in the design of columns.

Table A-17: Preliminary Column Design

Story Number	Column Section Areas for Min. Omega (mm^2)	Column Sections for Min. Omega	Unit Weight (kg/m)	Moment of Inertia(cm^4)
1	39920	HD 400 x 314	314.0	42600
2	39920	HD 400 x 314	314.0	42600
3	30090	HD 400 x 237	237.0	31040
4	30090	HD 400 x 237	237.0	31040
5	18790	HD 400 x 147	147.0	16720
6	18790	HD 400 x 147	147.0	16720
7	11250	HE 300 A	88.0	6310
8	11250	HE 300 A	88.0	6310

Table A-17: Preliminary Column Design-(continued)

Story Number	Column N_{CR} (kN)	Imperfection Factor (α)	Reduction Factor (χ)	Φ
1	52556	0.49	0.8321	0.7130
2	52556	0.49	0.8321	0.7130
3	38294	0.49	0.8271	0.7199
4	38294	0.49	0.8271	0.7199
5	20627	0.49	0.8038	0.7520
6	20627	0.49	0.8038	0.7520
7	7785	0.49	0.7146	0.8830
8	7785	0.49	0.7146	0.8830

After all members are designed, displacements at all stories are determined and they are given in Table A-18:

Table A-18: Displacements of the Structure

Story Number	Story Displacements (m)	Interstory Drifts (m)	Interstory Drift Ratio ($\delta R/h_s$)
1	0.0050	0.0050	0.50%
2	0.0102	0.0052	0.52%
3	0.0154	0.0052	0.52%
4	0.0208	0.0054	0.54%
5	0.0265	0.0057	0.57%
6	0.0321	0.0056	0.56%
7	0.0378	0.0056	0.56%
8	0.0425	0.0047	0.47%

Interstory drifts are within the limits stated in Eqn. 2.27.

The resulting structure has a period of 1.16 sec.

If the developed iterative method is used for this example, results would be as follows:

Design starts with a 1.16 sec initial period value. Base shear is determined as 304.81 kN by conducting the following calculations.

$T_C = 0.5,\ T_B = 0.15,\ T_1 = 1.16 \qquad T_C < T_1 < T_D$

$Eqn\ 2.3$ gives $\quad S_d(T) = 0.1 \times g \times 1.4 \times \dfrac{2.5}{4} \times \dfrac{0.5}{1.16} = 0.37\ \text{m/s}^2$

$T_1 > 2T_C \qquad \lambda = 1$

$F_b = 0.37 \times 79 \times 8 \times 1 \times 1.3 = 304\ \text{kN}$

Table A-19: Forces Acting on Stories and Columns Forces

Story Number	Forces Acting on Stories (kN)	Right Columns Forces (kN)	Left Columns Forces (kN)
1	8.5	-2293.5	-1480.7
2	16.9	-2066.1	-1253.2
3	25.4	-1671.8	-1187.9
4	33.9	-1439.5	-955.7
5	42.3	-1069.3	-856.5
6	50.8	-832.2	-619.4
7	59.3	-505.6	-466.9
8	67.7	-263.7	-225.0

Final brace and column design results are indicated in the following tables:

Table A-20: Final Brace Design

Story Number	Brace Forces (kN)	Omega Values (Ω)	Brace Section Areas (mm^2)	Brace Sections	Unit Weight (kg/m)
1	351.1	5.3694	5310	CHS x 219.1 x 8	41.6
2	341.3	5.5228	5310	CHS x 219.1 x 8	41.6
3	321.8	5.6479	5120	RHS x 150 x 200 x 8	40.2
4	292.6	5.5332	4560	RHS x 150 x 250 x 6	35.8
5	253.6	5.3764	3840	RHS x 150 x 250 x 5	30.1
6	204.8	5.5644	3210	CHS x 168.3 x 6.3	25.2
7	146.3	5.5575	2290	RHS x 140 x160 x 4	18.0
8	78.0	6.6890	1470	CHS x 159 x 3	11.5

Table A-20: Final Brace Design-(continued)

Story Number	Brace Yield Force (kN)	Brace N_{CR} (kN)	Brace Moment of Inertia (cm^4)	Brace Slenderness (λ)
1	1885.1	898.8	2959.6	1.45
2	1885.1	898.8	2959.6	1.45
3	1817.6	551.3	1815.5	1.82
4	1618.8	537.0	1768.3	1.74
5	1363.2	457.9	1508	1.73
6	1139.6	319.9	1053.4	1.89
7	813.0	220.4	725.6	1.92
8	521.9	135.9	447.4	1.96

Table A-21: Final Column Sizes and Properties

Story Number	Column Section Areas (mm^2)	Column Sections	Unit Weight (kg/m)	Moment of Inertia(cm^4)	Radius of Gyration (mm)
1	53710	HD 400 x 421	421.0	60080	10.58
2	53710	HD 400 x 421	421.0	60080	10.58
3	39920	HD 400 x 314	314.0	42600	10.33
4	39920	HD 400 x 314	314.0	42600	10.33
5	25030	HD 360 x 196	196.0	22860	9.56
6	25030	HD 360 x 196	196.0	22860	9.56
7	13350	HE 340 A	105.0	7436	7.46
8	13350	HE 340 A	105.0	7436	7.46

Table A-21: Final Column Sizes and Properties-(continued)

Story Number	Column N_{CR} (kN)	Imperfection Factor (α)	Reduction Factor (χ)	Φ	Column Slenderness (λ)
1	74121	0.49	0.8390	0.7039	0.507
2	74121	0.49	0.8390	0.7039	0.507
3	52556	0.49	0.8321	0.7130	0.519
4	52556	0.49	0.8321	0.7130	0.519
5	28202	0.49	0.8081	0.7461	0.561
6	28202	0.49	0.8081	0.7461	0.561
7	9174	0.49	0.7130	0.8854	0.719
8	9174	0.49	0.7130	0.8854	0.719

Table A-22: Final Column Forces and Capacities

Story Number	Column Forces (kN)	Column Capacities (kN)	Column Overstrengths
1	-2293.5	15997	6.97
2	-2066.1	15997	7.74
3	-1671.8	11793	7.05
4	-1439.5	11793	8.19
5	-1069.3	7180	6.71
6	-832.2	7180	8.63
7	-505.6	3379	6.68
8	-263.7	3379	12.81

Fundamental period of the structure is equal to 1.15 sec in final step.

Non-iterative method gives a total weight of 3388 kN/m while iterative method gives a total weight of 4388 kN/m.