

SEISMIC PERFORMANCE EVALUATION OF TWO-STOREY MOMENT-
RESISTING PRE-CAST CONCRETE INDUSTRIAL STRUCTURES

by

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to my family and colleagues

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ABSTRACT

SEISMIC PERFORMANCE EVALUATION OF TWO-STOREY MOMENT-RESISTING PRE-CAST CONCRETE INDUSTRIAL STRUCTURES

Turkey, as a country with majority of its soil prone to high seismic risk, did not only suffer from earthquakes in terms of life casualties, but it also suffered economically. Many industrial structures have been subjected to severe damage, especially in 1999 Marmara Earthquake. Considering an important amount of industrial structures in Turkey are made using pre-cast concrete technology, it is crucial to investigate seismic performance of pre-cast concrete industrial structures.

The buildings that are analyzed in this study possess several important aspects specific to themselves, including very large spans and loading, staged construction and differing connection conditions of columns at storey level and the roof.

In this study, two pre-cast buildings, with differing storey heights, are analyzed and designed in accordance with Turkish Seismic Code 2007 (TSC 2007). Seismic performance analyses of these two buildings are done using linear and nonlinear methods of TSC 2007. In light of the results obtained, design strategies are developed in order to improve seismic performance of this type of pre-cast industrial structures. The first strategy is to design the two frames using cracked section rigidities and the second strategy is to strengthen one-storey columns that cease at first floor.

Totally six frames, including the ones that are designed using developed performance improving strategies, are investigated analytically to determine their seismic performance.

Although it should be kept in mind that further and more detailed research is needed to support this study, the results obtained are interpreted to draw the following conclusions and recommendations:

- This type of pre-cast structures designed using uncracked section rigidities do not satisfy target seismic performance level (Life Safety Level) specified in TSC 2007.

However, same structures designed using cracked section rigidities satisfy target seismic performance level.

- Strengthening one-storey columns prevents plastic behaviour to accumulate in their sections and improves overall seismic performance of the structure.
- Nonlinear static analysis methods do not yield reliable results for this type of structures and should be supported by nonlinear dynamic analysis methods.
- Since design of beams is governed by very large gravitational loading and the connections of pre-cast frames are designed considering amplified seismic loads, beams perform well in this type of pre-cast structures. The seismic performance of these buildings is governed by behaviour of columns.

ÖZET

İKİ KATLI MOMENT AKTARAN BAĞLANTILI PREFABRİK ENDÜSTRİ YAPILARININ DEPREM PERFORMANSININ İNCELENMESİ

Topraklarının büyük bir kısmı deprem riski altında olan Türkiye, depremlerden sadece can kaybı olarak değil ekonomik açıdan da büyük zarar görmüştür. Özellikle 1999 Marmara Depremi'nde birçok endüstri yapısında ciddi hasar görülmüştür. Ülkemizdeki endüstri yapılarının büyük bir kısmının prefabrike beton teknolojisi kullanılarak inşa edildiği göz önünde bulundurulduğunda, bu tip binaların deprem performansının belirlenmesinin önemi açıktır.

Bu çalışmada incelenen prefabrike binaların, büyük açıklık ve yükler, aşamalı yapım ve kolonların kirişlere kat seviyesinde moment aktaran bağlantılı, çatıda ise mafsallı olması gibi kendine has özellikleri bulunmaktadır.

Çalışmada, değişen kat yüksekliklerine sahip iki prefabrike endüstri binası 2007 Türk Deprem Yönetmeliği'ne göre tasarlanmış ve deprem performansı belirlenmiştir. Deprem performans analizinden elde edilen sonuçlara göre binaların deprem performansının tasarım aşamasında iyileştirilmesine yönelik iki ana strateji geliştirilmiştir. Bu stratejilerden biri, çatlamamış kesit rijitliklerine göre tasarlanan binaların çatlamış kesit rijitlikleri dikkate alınarak tasarlanması, bir diğeri ise çatıya çıkmayan kolonların taşıma gücünün artırılmasıdır.

Geliştirilen bu stratejilerle tasarlanan toplam altı adet çerçevenin, Türk Deprem Yönetmeliği 2007'ye göre performans analizi yapılmıştır. Daha sonra elde edilen sonuçlardan faydalanılarak çeşitli sonuçlara varılmıştır.

Elde edilen sonuçların daha güvenilir olması için daha detaylı ve daha fazla çerçeve tipini içeren analitik çalışmaların yapılması gerektiği açıktır. Bu çalışmanın sonuçları ışığında varılan sonuçlar şöyledir:

- Çatlamamış kesit rijitlikleri kullanılarak tasarlanan bu tip prefabrik endüstri binaları Türk Deprem Yönetmeliği 2007’de belirtilen Performans Hedefi olan Can Güvenliği Performans Düzeyi’ni sağlamamaktadır. Çatlamış kesit rijitlikleri kullanılarak tasarlanan aynı binalar ise hedeflenen performans düzeyini sağlamaktadır.
- Çatıya çıkmayan ve birinci katta biten kolonların taşıma güçlerinin arttırılması, plastik davranışın bu kesitlerde yoğunlaşmasını engellemekte ve binanın deprem performansını arttırmaktadır
- Bu tip binaların performans analizlerinde doğrusal olmayan statik analiz yönteminin kullanılması güvenilir değildir. Bu yöntemle elde edilen sonuçların, doğrusal olmayan dinamik yöntemlerle elde edilen sonuçlarla desteklenmesi gerekmektedir.
- Bu tip binaların kirişlerinin tasarımı genellikle büyük düşey yüklere göre belirlendiğinden ve bu kirişlerin taşıma güçleri Türk Deprem Yönetmeliği 2007’de prefabrike binaların bağlantılarının tasarımında deprem yüklerinin arttırılması şartı dolayısıyla oldukça büyük olduğundan, kirişlerin performansı yeterli seviyededir. Bir başka deyişle, bu tip binalarda binanın performansını kolonların davranışı belirlemektedir.

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LIST OF SYMBOLS

A_s/s_{prov}	: Provided transverse reinforcement area / distance (cm)
A_s/s_{req}	: Required transverse reinforcement area / distance (cm)
EI_e	: Effective section rigidity
EI_o	: Uncracked section rigidity
a	: Depth of compression block
$A(T)$	: Spectral Acceleration Coefficient
A'_s	: Compression reinforcement area
$a_1^{(i)}$	: Modal acceleration at i^{th} step of pushover analysis
A_c	: Total cross-sectional area of column
A_{ck}	: Total area inside transverse reinforcement that confines concrete
A_o	: Effective Ground Acceleration Coefficient
A_{ps}	: Pre-stressed steel area
A_s	: Non-prestressed steel area
$A_{s,prov}$	: Provided longitudinal reinforcement area (cm ²)
$A_{s,req}$	: Required longitudinal reinforcement area (cm ²)
A_{sh}	: Area of transverse reinforcement required
b	: Width of the section
b_k	: Axial distance between transverse reinforcement that confines concrete
b_w	: Web width
d	: Effective depth of the section
$d_1^{(i)}$	: Modal displacement at i^{th} step of pushover analysis
d_{fi}	: Displacement of i^{th} storey subject to F_{fi}
E_p	: Modulus of elasticity of steel
f'_{pd}	: Design effective pre-stressed steel strength
f_{cc}	: Confined concrete strength in Mander Model
f_{cd}	: Concrete compressive strength
f_{cjk}	: Compressive strength of concrete at j^{th} day
f_{ck}	: 28-days compressive strength of concrete
f_{cm}	: Existing material strength for concrete

f_{co}	: Compressive strength of unconfined concrete in Mander Model
f_{ctd}	: Design tensile strength of concrete
f_e	: Effective confining stress in Mander Model
F_{fi}	: Fictive load acting on i^{th} storey
F_i	: Seismic force acting on i^{th} storey
f_p	: Steel stress
f_{pb}	: Steel stress at point B
f_{pc}	: Steel stress at point C
f_{pd}	: Design pre-stressed steel strength
f_{pu}	: Ultimate steel stress
f_{yd}	: Design yield strength of non-prestressed steel
f_{yd}	: Design yield strength of reinforcing steel
f_{ywk}	: Yield strength of transverse reinforcement
g_i	: Weight of i^{th} storey due to permanent loads
h	: Height of reinforced concrete section
h_i	: Height of i^{th} storey
I	: Building Importance Factor
L_p	: Plastic hinge length
$m_{\Theta i}$	: For buildings at which slabs act as rigid diaphragms, mass moment of inertia about vertical axis
m_i	: Mass of i^{th} storey
M_{ra}, M_{ru}	: Ultimate strength of the column section below and above the connection, respectively
M_{res}	: Ultimate strength of the section
M_{ri}, M_{rj}	: Ultimate strength of the beam section left and right of the connection, respectively
M_{x1}	: Effective mass of building at earthquake in x-direction corresponding to first natural vibration mode
n	: Live load participation factor, as defined by TSC2007 Table 2.7
N_{dm}	: Maximum axial force due to combined effects of gravitational and seismic loads
q_i	: Weight of i^{th} storey due to live loads
r	: Demand/capacity ratio

R	: Structural behaviour factor
R_a	: Reduced Structural Behaviour Factor
R_{bottom}	: Structural behaviour factor at storey level
r_s	: Demand/capacity ratio limit
R_{top}	: Structural behaviour factor at roof level
s	: Longitudinal distance between transverse reinforcement
$S(T)$	: Spectrum Coefficient
S_{di1}	: Nonlinear spectral displacement
t	: Flange thickness
T_1	: First natural vibration period
T_A, T_B	: Spectrum Characteristic Periods
$u_{x1}^{(i)}$	: Tip displacement of building at i^{th} step of pushover analysis corresponding to first vibration mode
V_E	: Shear force considered when determining shear capacity of ductile member
V_i	: Seismic shear force acting on i^{th} storey
$V_{x1}^{(i)}$	: Base shear at i^{th} step of pushover analysis
W	: Total weight of the building
x	: Normalized concrete strain in Mander Model
$(\Delta_i)_{\text{ort}}$	: Average reduced relative storey drift at i^{th} storey
$(\delta_i)_{\text{max}}$	: Maximum effective relative storey drift
ΔF_N	: Additional seismic force acting on n^{th} storey of the building
Θ_p	: Plastic rotation
γ_{mc}	: Material coefficient for cast-in-situ concrete
γ_{mpc}	: Material coefficient for pre-cast concrete
γ_{mps}	: Material coefficient for pre-stressed steel
γ_{ms}	: Material coefficient for non-prestressed steel
ϵ_p	: Steel strain
ϵ_{pb}	: Steel strain at point B (the limit of proportionality)
ϵ_{pc}	: Steel strain at point C
ϵ_{pu}	: Ultimate steel strain
ρ	: Longitudinal tensile reinforcement area
ρ'	: Longitudinal compressive reinforcement area

ρ_b	: Longitudinal balanced reinforcement area
ρ_p	: Pre-stressed steel ratio
ρ_x	: Volumetric ratio of transverse reinforcement in x-direction
ρ_y	: Volumetric ratio of transverse reinforcement in y-direction
σ_{pof}	: Effective stress at pre-stressed steel after losses
$\phi_{\Theta i1}$	: For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey about vertical axis
ϕ_p	: Plastic curvature demand
ϕ_{prov}	: Provided rebars
ϕ_t	: Total curvature demand
ϕ_{xi1}	: For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey in x direction
ϕ_y	: Yield curvature
ϕ_{yi1}	: For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey in y direction

LIST OF ACRONYMS/ABBREVIATIONS

MDL	: Minimum Damage Level
MDR	: Minimum Damage Region
SDL	: Significant Damage Level
SDR	: Significant Damage Region
VDL	: Visible Damage Level
VDR	: Visible Damage Region
TSC2007	: Turkish Seismic Code 2007

1. INTRODUCTION

1.1 General

The Anatolian soil, which constitutes the majority of Turkey, is prone to high seismic risk. Turkey has experienced strong earthquakes with catastrophic impacts, including but not limited to 1939 Erzincan Earthquake, which left 32,700 dead, 1966 Varto Earthquake with 2,529 fatalities, and more recently 1999 Izmit Earthquake which caused 17,118 people to die [1].

These earthquakes had a damaging effect not solely in terms of body count, but they also undermined the country's economy significantly. Many industrial structures have been severely damaged in the 1999 Izmit Earthquake [2]. Considering an important amount of industrial buildings in Turkey are constructed with pre-cast members, it is vital to investigate seismic performance of pre-cast concrete industrial structures in Turkey.

Among those pre-cast concrete industrial structures in Turkey, vast majority are one-storey buildings with or without partial mezzanines. Research on this type of pre-cast industrial buildings, also known as “inverted pendulum” structures, is present in literature.

However, although not as common as one-storey pre-cast industrial buildings, multi-storey pre-cast concrete buildings are also observed in Turkey. This type of structures are widely used as warehouse buildings or manufacturing facilities. Although there are site investigations done examining damages caused by the 1999 Izmit Earthquake on multi-storey pre-cast buildings, there is limited amount of analytical research on the seismic performance of this kind of pre-cast buildings. The need for analytical research on multi-storey pre-cast structures becomes more clear considering special design and construction aspects of pre-cast concrete structures.

1.2 Special Aspects of Pre-cast Concrete Construction

Analysis of pre-cast concrete structures requires considering several aspects specific to pre-cast construction. These specific aspects are listed below:

(i) Staged Construction

Analysis of pre-cast concrete structures requires considering two stages of construction. To understand the two main stages in pre-cast construction more clearly, it is important to acknowledge how pre-cast members are erected and their connections are established. Firstly, pre-cast columns are placed inside cast-in-situ socket foundations. Secondly, pre-cast storey beams are erected and their connections with the columns are established by welding the plates embedded at the bottom of the beams to those embedded at the top of the column corbels. (Fig 1.1) Secondly, pre-cast TT slabs, supported by the web corbels of the beams are erected. At the roof, main roof girders, connected to the column corbels by pinned connections, are erected. (Fig 1.2) Pre-cast roof purlins, constituting gravitational load carrying system for the roof, are also connected to the main girders by pinned connections. Following this stage, continuity rebars, which resist negative moment at column-beam connection, are placed passing along the voids left at the columns at manufacturing. (Fig 1.3) Finally, topping concrete is cast and frame behaviour is established. Composite behaviour between pre-cast members and topping concrete is ensured by tie reinforcement protruding from pre-cast members. (Fig 1.4)

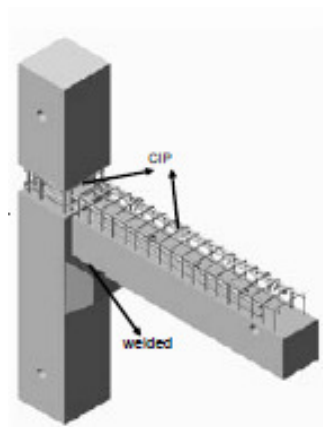


Figure 1.1. Beam-column connection [3].



Figure 1.2. Roof girder-column connection [4].

As can be seen in Figure 1.2, roof girders are erected on column corbels by the help of pins extending from the column corbel. This kind of connection is assumed to be a hinged connection.

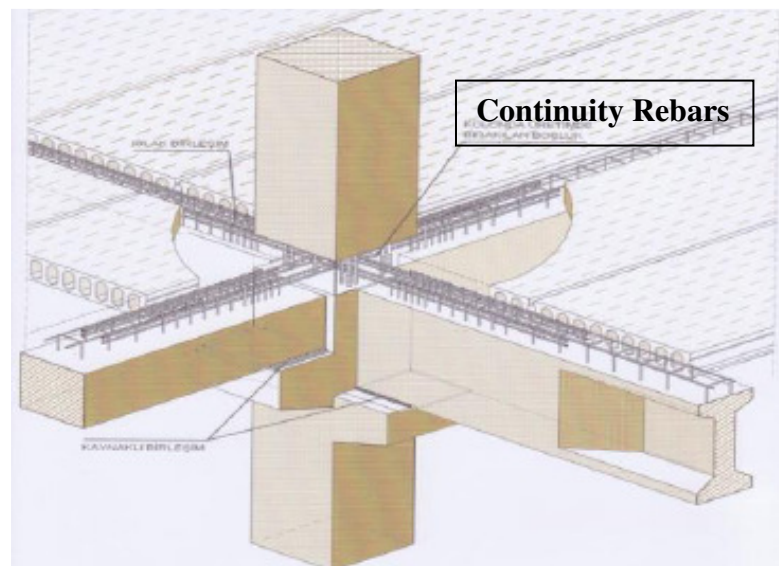


Figure 1.3. Continuity rebars at column-beam connections [4].

Figure 1.3 illustrates the moment-resisting column-beam connection at storey level. Continuity rebars pass through the voids left at the columns at first stage after the beams are erected. Continuity rebars are placed inside the tie bars extending from the beams. After the continuity rebars are placed, 150mm thick cast-in-situ concrete is poured to establish composite behaviour and diaphragm action.

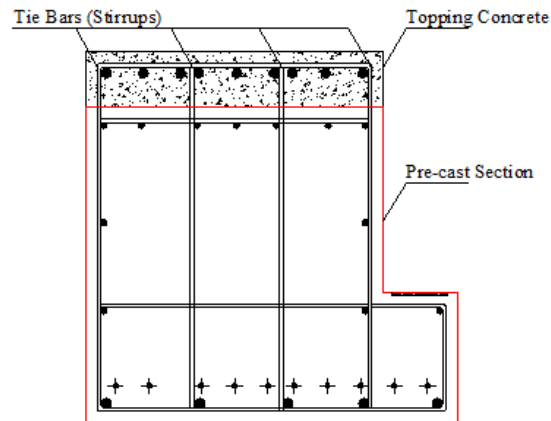


Figure 1.4. Tie bars in pre-cast storey beams.

These two stages need to be considered when analyzing pre-cast concrete structures. The first one is the stage in which pre-cast columns are placed inside cast-in-situ socket foundations and pre-cast beams and slabs are erected. At this stage, continuity between pre-cast main storey beams and thus frame behaviour is not yet established. The loads due to self weight of beams and slabs as well as that of the topping concrete act eccentrically on the columns. Load of the topping concrete also needs to be considered to act at the first stage because pre-cast section, rather than composite section, resists self weight of topping concrete before the concrete hardens and composite behaviour is established. Briefly, the first stage is the stage in which ‘cantilever’ columns resist eccentric gravitational loads due to self weight of the main storey beams, slabs, topping concrete and roof members.

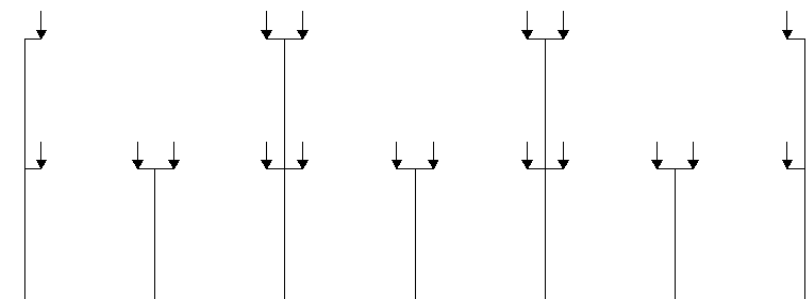


Figure 1.5. Structural behaviour at stage one.

The second stage is when topping concrete hardens and connection between pre-cast members and thus frame behaviour is established. Live loading and seismic loads act on the frame at this stage.

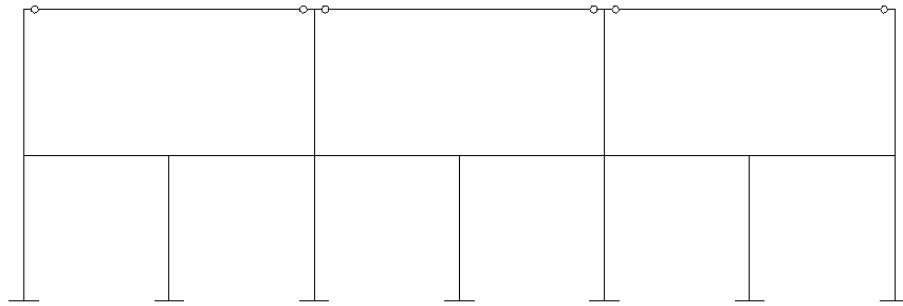


Figure 1.6. Structural behaviour at stage two.

(ii) Pre-cast columns are hinged at the roof

Another specific aspect of pre-cast construction is that the columns are rigidly connected to frame members at the main storey and have hinged connections with the roof girders at the top. This aspect requires a special approach for Earthquake Design, considering differing Structural Behaviour Factor (R) values defined at Turkish Seismic Code 2007 (TSC 2007) Table 2.5.

Table 1.1. Structural behaviour factors (R) for pre-cast structures as per TSC2007.

Building Structural System	Systems of Nominal Ductility Level	Systems of High Ductility Level
Prefabricated Reinforced Concrete Buildings		
Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	7
Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases	-	3
Buildings in which seismic loads are fully resisted by prefabricated solid structural walls	-	5
Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	6

As can be seen at Table 1.1 above, *Structural Behaviour Factor R* for ‘pre-cast buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer’ with a High Ductility Level is 7, whereas *Structural Behaviour Factor R* for ‘pre-cast buildings in which seismic loads are fully resisted by single storey hinged frames with fixed-in bases’ is 3.

TSC 2007 Section 2.5.5.3 states that when ‘single-storey frames with columns hinged at the top are used as top (roof) storey of cast-in-situ, pre-cast or steel buildings, R values defined at Table 2.5 for top (R_{top}) and intermediate stories (R_{bottom}) can be used together’, provided:

- At first stage, earthquake analysis shall be performed according to 2.7 or 2.8, taking R equal to R_{bottom} and storey drifts shall be determined in conjunction with this analysis
- Stress resultants for the top storey shall be determined by multiplying the effects calculated by first stage by R_{bottom}/R_{top}
- Stress resultants for the bottom part (intermediate storeys) shall be determined by the sum of two parts, first of which is the stress resultants determined at first stage. In second part, the stress resultants are determined by multiplying the top storey support reactions by $(1-R_{top}/R_{bottom})$ and applying these reactions to the bottom part (intermediate stories).

A special algorithm developed by Ozmen is used in the analysis of the pre-cast structures. This algorithm is described in detail in Chapter 2.

(iii) Moment resisting connections between pre-cast members

Connections between members are inarguably the ‘weakest link’ in pre-cast construction which require special attention. Establishing moment transfer between pre-cast members is crucial. Aside the fact that skilled labour and strict quality control is essential to ensure correct application/construction of the connection, such connections should be very carefully analyzed and designed.

TSC 2007 Section 3.12.2.2 states that ‘connections shall possess sufficient strength to transfer moments, shear forces and axial forces to be developed at the ultimate strength level without any reduction in strength and ductility. In welded connections and other types of connections, 2 times and 1.5 times the seismic connection forces, respectively, obtained according to Chapter 2 shall be taken into account’.

Moment transfer in beam-column connections of pre-cast members analyzed in this study is established by force couples. Negative moments are resisted by longitudinal reinforcement passing through the voids left at the columns at fabrication phase. Force transfer for positive moments is established with welding of embedded plates in beams and columns. Elements of a typical moment-resisting pre-cast connection is shown in Figure 1.7 below. The adequacy of this type of connections is studied by Ertas et al. and presented in Section 2.3 of this study.

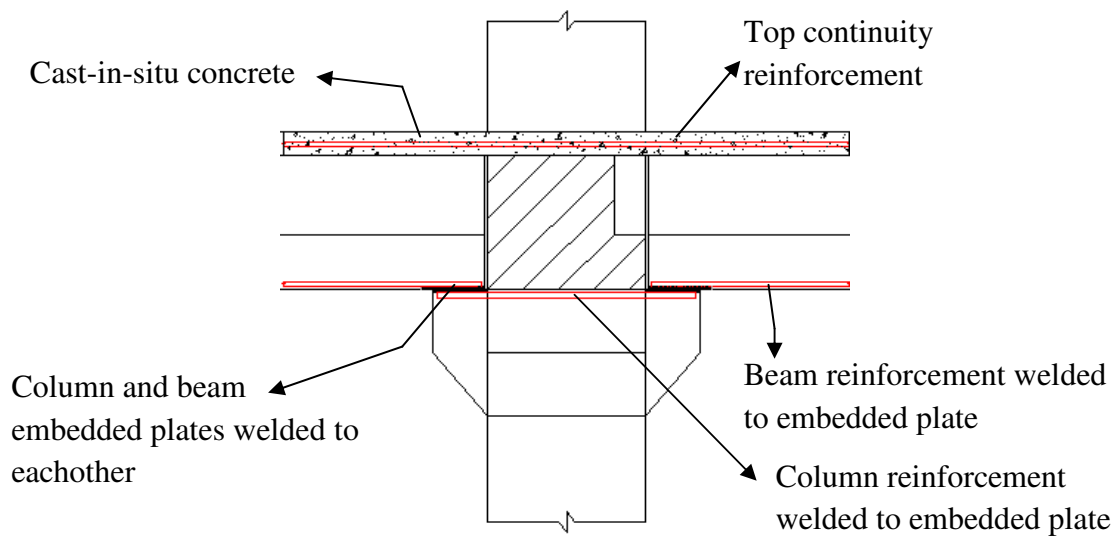


Figure 1.7. Force transfer mechanism at moment-resisting pre-cast connections.

1.3 Scope of the study

The analytical research study that is presented in this document is briefly the investigation of the seismic performance of typical pre-cast frames (Fig 1.8) with moment resisting connections. Storey height h is 10.0 m for Building 1-1A and 1B and 7.0m for Building 2-2A and 2B.

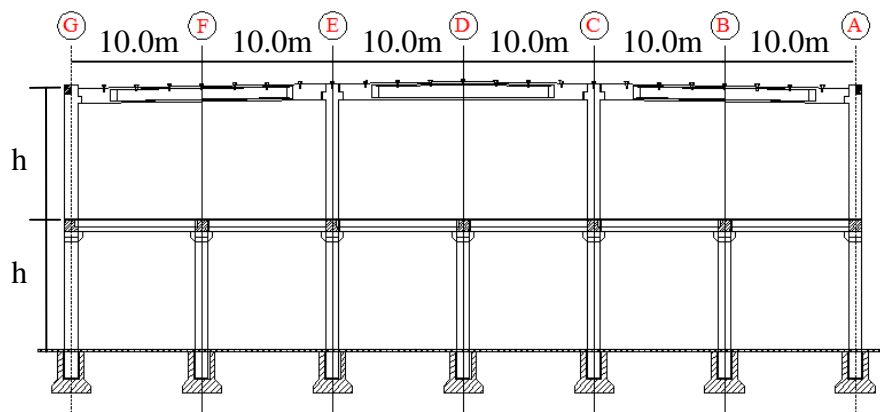


Figure 1.8. Typical frame of the pre-cast buildings analyzed.

Major steps of this study are listed below:

- Analysis and design of two typical frames, with varying storey heights, consisting of pre-cast members with moment resisting connections as per Turkish Seismic Code 2007 Sections 2 and 3,
- Performance analysis of the two frames using Linear Elastic Method as per Turkish Seismic Code 2007 Section 7.5,
- Performance analysis of the two frames using Nonlinear Static (Pushover) Method as per Turkish Seismic Code 2007 Section 7.6,
- Evaluating the results determined by the two different performance analysis methods for the two frames,
- Developing strategies to improve performance levels of these two frames,
- Analyzing seismic performance of the two frames re-designed utilizing the strategies developed,

- Evaluating the adequacy of the strategies developed in light of the results obtained and proposing recommendations to improve seismic performance of such pre-cast structures at initial design stage.

2. LITERATURE SURVEY

A comprehensive literature survey about this specific type of pre-cast structures is presented below:

2.1 Moment-Curvature Analysis and Ductility of Partially Pre-stressed Beams

TSC 2007 Section 3.12.3 states that ‘with the exception of floor elements and girder-type elements with hinged connections to the columns of single-storey buildings, full pre-stressing shall not be permitted in prefabricated structural elements to be constructed in seismic zones. Appropriate amount of non-prestressed steel reinforcement shall be used to achieve sufficient ductility.’

An experimental research on ductility of partially pre-stressed beams is done by Thompson and Park [5]. The authors emphasize that:

“The suspicions that exist concerning prestressed concrete in seismic design are generally a concern whether the energy dissipation at the plastic hinge regions is adequate, and whether such sections can achieve the required ductility. The first issue arises because for prestressed concrete members the initial elastic tensile strain in the tendons due to prestress causes a large deflection recovery, even after large deflections. Hence, the energy dissipation (area within the loop) of prestressed concrete members will be lower than that of reinforced concrete members with similar strength” [5].

Thompson and Park also state that:

“The moment-curvature loops of of prestressed members can be ‘fattened’, and hence the displacement response reduced, if non-prestressed steel is added to the member to provide energy dissipation. The presence of longitudinal non-prestressed reinforcing steel also improves ductility by acting as compression reinforcement” [5].

Thompson and Park developed a moment-curvature analysis procedure for partially pre-stressed members that ensures compatibility of strains and equilibrium of forces and is based on idealizations for the stress-strain behaviour of concrete and steel [5].

The stress-strain model for pre-stressing steel used by Thompson and Park in their study involves three regions. (Fig 2.1)

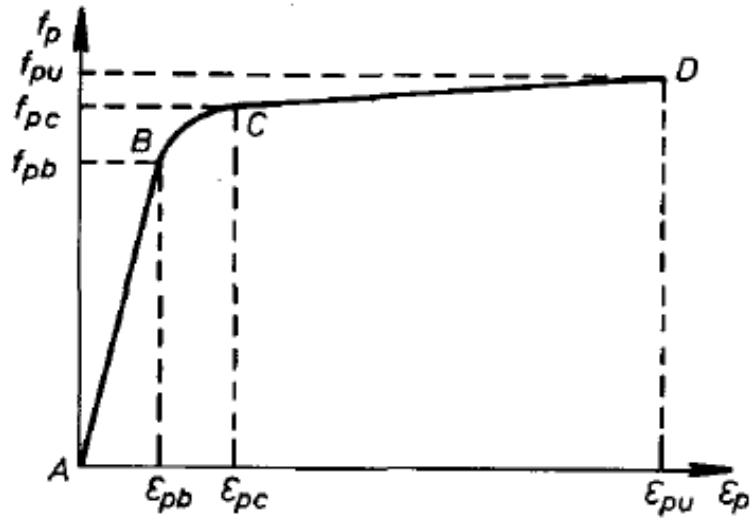


Figure 2.1. Stress-strain model for pre-stressing steel used by Thompson and Park [5].

The stress-strain relations for pre-stressing steel for the three regions are given by:

Region AB ($\epsilon_p \leq \epsilon_{pb}$):

$$f_p = E_p \epsilon_p \quad (2.1)$$

in which

ϵ_p = steel strain

ϵ_{pb} = steel strain at point B (the limit of proportionality)

f_p = steel stress

E_p = modulus of elasticity of steel

Region BC ($\epsilon_{pb} < \epsilon_p \leq \epsilon_{pc}$)

$$f_p = \frac{f_{pc} \epsilon_{pc} - f_{pb} \epsilon_{pb}}{\epsilon_{pc} - \epsilon_{pb}} + \frac{\epsilon_{pb} \epsilon_{pc} (f_{pb} - f_{pc})}{\epsilon_p (\epsilon_{pc} - \epsilon_{pb})} \quad (2.2)$$

in which

ϵ_{pc} = steel strain at point C

f_{pb} = steel stress at point B

f_{pc} = steel stress at point C

Region CD ($\epsilon_{pc} < \epsilon_p \leq \epsilon_{pu}$)

$$f_p = f_{pc} + \frac{\varepsilon_p - \varepsilon_c}{\varepsilon_{pu} - \varepsilon_{pc}} (f_{pu} - f_{pc}) \quad (2.3)$$

in which

ε_{pu} = ultimate steel strain

f_{pu} = ultimate steel stress

Thompson and Park state that:

“At all stages the stress in each pre-stressing tendon is that corresponding to a total steel strain which is the sum of the steel strain due to prestress when the adjacent concrete strain is zero plus the concrete strain which exists at the level of the steel” [5].

The stress-strain model used for concrete by Thompson and Park is Kent and Park model, which does not change with the existence of pre-stressing steel. Therefore, in this study, stress-strain models for concrete and non-prestressed steel is taken from TSC 2007 Appendix 7B.

Naaman performed an experimental study about moment-curvature analysis of partially prestressed sections as well [7]. Naaman states that:

“In order to determine maximum moment capacity the moment versus curvature curve is plotted and the peak value selected. This can be achieved numerically in the most general case first by selecting a value of concrete extreme fiber compressive strain and then finding by iteration a location of neutral axis which simultaneously satisfies compatibility and equilibrium. The process is repeated for increasing values of the extreme fiber compressive strain. By calculating at every step the internal moment and the curvature, the ultimate (maximum) moment and the corresponding strain and curvature at ultimate can be determined” [7].

“The above exact method was applied to prestressed concrete sections and was found to lead to conditions at ultimate not significantly different from a less exact method described below” [7].

2.2 Columns Hinged at the Top and Rigidly Connected at Storey Level

TSC 2007 Section 2.5.5.3 requires amplification of top storey forces to account for the change in R value for buildings whose top story columns are hinged at the top, as was described in Section 1.2.(ii). However, the approach given in TSC 2.5.5.3 is time-consuming. Ozmen [6] proposes a simpler method which yields identical results. Ozmen’s method, which is adopted in this study, involves the following two steps:

- Calculating seismic loads taking R equal to R_{bottom}
- Applying to the top storey an additional seismic load determined by multiplying the initial seismic load (determined in step 1) by $((R_{\text{bottom}}/R_{\text{top}})-1)$

2.3 Experimental Research on Moment-Resisting Pre-cast Column-Beam Connections

Ertas et al [3] performed an experimental study to investigate ductility of pre-cast moment resisting column-beam connections. The authors tested five specimens, one of which is monolithic specimen, each representing different types of connections used in modern practice. One of these specimens, namely ‘Composite Connection’, is identical to the type used in this study (Figure 2.2).

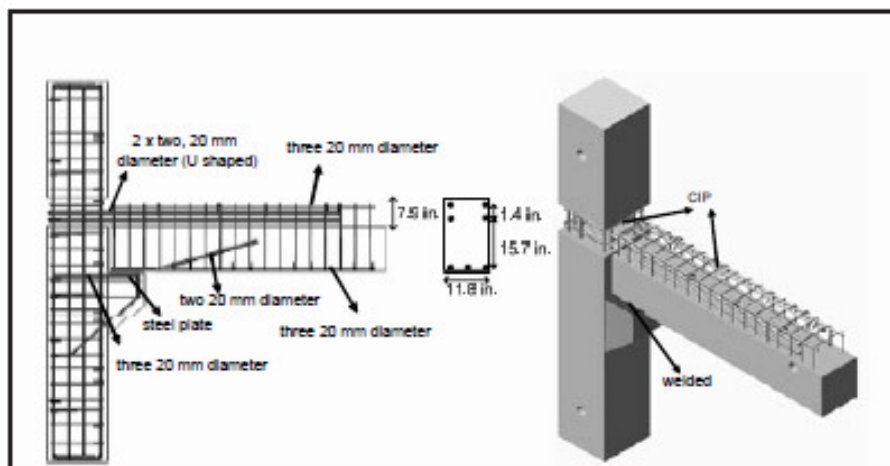


Figure 2.2. Composite connection used in the study of Ertas et al [3].

The authors describe the specimen as:

“The composite connection type ensured the continuity of the beam’s bottom reinforcement by welding and the top reinforcement by placing cast-in-place concrete through the gap in the column. This is a common connection used by Turkish precast concrete producers” [3].

The performance of the five specimens subject to displacement control reverse cyclic loading is tested and compared with the monolithic connection. Only the results obtained for the Composite Specimen shall be presented here.

The observed behaviour of the Composite Specimen is as follows:

- The first flexural crack in the beam was observed at the 0.5% story drift level and was located 250mm away from the precast concrete column
- At the 1.4% story drift level, a diagonal crack was observed at the corbel-column region
- Diagonal cracking at the beam-column joint core was first obtained at 2.2% story drift.
- The failure of specimen occurred suddenly with the rupture of the beam's bottom reinforcement at the 3.5% story drift level
- The early rupture of the reinforcement may well be explained with the changing mechanical properties of the steel due to the welding done during the preparation of steel cages prior to molding.

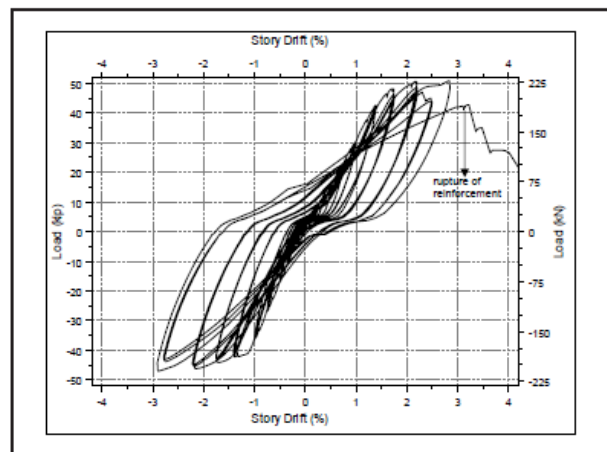


Figure 2.3. Lateral load vs storey drift response of composite connection specimen.



Figure 2.4. Damage in composite connection specimen at 3.5% storey drift level.

Ertas et al [3] evaluated the test results as follows:

- All connections reached their calculated ultimate and yield moment strengths. The capacity prediction for the backward cycle of composite connection specimen was less than the experimentally measured value due to the existence of the column corbel, which served as a haunched beam end
- There was no significant stiffness degradation in composite connection specimen up to the 1% storey drift level. At 2.75% storey drift, approximately 50% of the initial stiffness was reserved
- All tested precast concrete connections are suitable for high seismic zones in terms of strength properties and energy dissipation
- Composite connection specimen with welding yielded an inferior performance compared with other types of specimens
- All connections reached their calculated ultimate and yield moment capacities
- Composite connection specimen could not sustain up to 3.5% storey drift level, as opposed to the other specimens. Excessive welding may adversely affect the mechanical properties of the reinforcement and is believed to be the cause of the inferior performance of composite connection specimen

2.4 Analysis and Design of Pre-stressed Members

Requirements regarding analysis and design of pre-stressed members are specified in TS-3233 [9], Turkish Pre-stressed Concrete Code. Relevant specifications are listed below:

- TS-3233 8.4.3 states that material coefficients to be used in ultimate strength analysis of members are:

$$\gamma_{mc} : 1.5$$

$$\gamma_{mpc} : 1.4$$

$$\gamma_{ms} : 1.15$$

$$\gamma_{mps} : 1.15$$

in which

$$\gamma_{mc} : \text{Material coefficient for cast-in-situ concrete}$$

$$\gamma_{mpc} : \text{Material coefficient for pre-cast concrete}$$

γ_{ms} : Material coefficient for non-prestressed steel

γ_{mps} : Material coefficient for pre-stressed steel

- Table 2.1 shows allowable stresses for pre-stressed members as specified in TS-3233 Section 9.

Table 2.1. Allowable stresses for pre-stressed members as specified in TS-3233.

	Compression (MPa)		Tension(MPa)	
At transfer stage	Members fabricated at facility	$0.60f_{cjk}$	At supports of simply supported members	$0.50\sqrt{f_{cjk}}$
	Members fabricated at site	$0.55f_{cjk}$	Other members	$0.25\sqrt{f_{cjk}}$
At service stage	Bridge members	$0.40f_{ck}$	Fully pre-stressed members	$0.50\sqrt{f_{ck}}$
	Other members	$0.45f_{ck}$	Partially pre-stressed members	$1.00\sqrt{f_{ck}}$

In Table 2.1 above;

Transfer stage: The moment in which pre-stressing strands are released

Service stage: The stage in which service (long-term) loads act

f_{ck} : 28-days compressive strength of concrete

f_{cjk} : Compressive strength of concrete at j^{th} day

- TS-3233 specifies that in order to achieve sufficient ductility in members to be constructed in seismic zones;
 - (i) Partial pre-stressing should be used. Partial pre-stressing is established by using continuous non-prestressing reinforcement that constitutes at least 10% of the ultimate strength and limiting the ratio of concrete block depth to effective depth to 0.20.
 - (ii) The ratio of ultimate strength to cracking moment should be equal to or larger than 1.33.
- Ultimate strength of pre-stresses members as per TS-3233 Appendix C.1 is given by the following expressions:

(i) Ultimate Strength for rectangular members or flanged members for which compression block falls within the flange;

$$M_{res} = (A_{ps} f'_{pd} + A_s f_{yd}) (d - \frac{a}{2}) \quad (2.4)$$

$$a = \frac{A_{ps} f'_{pd} + A_s f_{yd}}{0.85 f_{cd} b} \quad (2.5)$$

$$f'_{pd} = f_{pd} (1 - 0.4 \rho_p \frac{f_{pd}}{f_{cd}}) \leq \sigma_{pof} + 40 MPa \quad (2.6)$$

In the expressions above;

- M_{res} : Ultimate strength of the section
- A_{ps} : Pre-stressed steel area
- f'_{pd} : Design effective pre-stressed steel strength
- A_s : Non-prestressed steel area
- f_{yd} : Design yield strength of non-prestressed steel
- d : Effective depth of the section
- a : Depth of compression block
- b : Width of the section
- f_{pd} : Design pre-stressed steel strength
- f_{cd} : Concrete compressive strength
- ρ_p : Pre-stressed steel ratio
- σ_{pof} : Effective stress at pre-stressed steel after losses

(ii) Ultimate Strength for flanged members for which compression block does not fall within the flange;

$$M_{res} = 0.85 f_{cd} [t(b - b_w)(d - t/2) + ab_w(d - a/2)] \quad (2.7)$$

$$a = \frac{A_{ps} f'_{pd} + A_s f_{yd} - 0.85 f_{cd} (b - b_w) t}{0.85 f_{cd} b_w} \quad (2.8)$$

in which

t : Flange thickness

b_w : Web width

(iii) Ultimate strength for doubly-reinforced rectangular member;

$$M_{res} = 0.85 f_{cd} b_w a (d - a/2) + A'_s f_{yd} (d - d') \quad (2.9)$$

$$a = \frac{A_{ps} f'_{pd} + f_{yd} (A_s - A'_s)}{0.85 f_{cd} b_w} \quad (2.10)$$

in which

A'_s : Compression reinforcement area

2.5 Seismic Analysis and Design According to Turkish Seismic Code 2007

In this study, seismic analysis and design principles of Turkish Seismic Code 2007 (TSC2007) [10] are used. Specifications of TSC2007 relevant to this study shall be presented in this section.

- Building Irregularities

Table 2.2. Building irregularities as per TSC2007 Table 2.1 [10].

A-IRREGULARITIES IN PLAN	Relevant Clauses
A1 – Torsional Irregularity: The case where <i>Torsional Irregularity Factor</i> η_{bi}, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2. $(\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} > 1.2)$ Storey drifts shall be calculated according to 2.7 considering the effects of 5% additional eccentricities	2.3.2.1

Table 2.2cont. Building irregularities as per TSC2007 Table 2.1 [10].

A-IRREGULARITIES IN PLAN	Relevant Clauses
A2 – Floor Discontinuities: In any floor: 1- The space where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area. 2- The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements 3- The cases of abrupt reductions in the in-plane stiffness and strength of floors	2.3.2.2
A3 – Projections in Plan: The cases where projections beyond the re-entrant corners in both of the two principle directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%	2.3.2.2
B-IRREGULARITIES IN ELEVATION	
B2-Interstorey Stiffnes Irregularity (Soft Storey): The case where in each of the two orthogonal earthquake directions. <i>Stiffness Irregularity Factor</i> η_{ki} , which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above, is greater than 2.0. $[\eta_{ki} = (\Delta_i)_{ort} / (\Delta_{i+1})_{ort} > 2.0]$ <i>Storey drifts shall be calculated in accordance with 2.7 by considering the effects of 5% additional eccentricities.</i>	2.3.2.1
B3-Discontinuity of Vertical Structural Elements: The cases where vertical structural elements (columns or structural walls) are removed at same stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath.	2.3.2.4

- Definition of Elastic Seismic Loads (Spectral Acceleration Coefficient)

The *Spectral Acceleration Coefficient*, $A(T)$, corresponding to 5% damped elastic *Design Acceleration Spectrum* normalized by the acceleration of gravity, g , is given by the following expression as per TSC2007 Eq. 2.1:

$$A(T) = A_o IS(T) \quad (2.11)$$

$$S_{ae}(T) = A(T)g \quad (2.12)$$

Notations above are described in the following tables.

- Effective Ground Acceleration Coefficient, A_o

Effective Ground Acceleration Coefficient, A_o , is defined in TSC2007 Table 2.2.

Table 2.3. Effective ground acceleration coefficient, A_o .

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

- Building Importance Factor, I

Building Importance Factor, I , is defined in TSC2007 Table 2.3.

Table 2.4. Building Importance Factor, I .

Purpose of Occupancy or Type of Building	Building Importance Factor
1. Buildings to be utilized immediately after the earthquake and buildings containing hazardous materials a) Buildings required to be immediately after the earthquake. (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities, governate, country and municipality administration buildings, first aid and emergency planning stations) b) Buildings containing or storing toxic, explosive and flammable materials, etc	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons etc. b) Museums	1.4
3. Intensely but short-term occupied buildings Sport facilities, cinema, theatre and concert halls etc	1.2
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures etc)	1.0

- Spectrum Coefficient, $S(T)$

Spectrum Coefficient, $S(T)$, is given by the expressions below as defined in TSC2007 Eq.2.2:

$$S(T) = 1 + 1.5 \frac{T}{T_A} \quad \text{for } 0 \leq T \leq T_A \quad (2.13)$$

$$S(T) = 2.5 \quad \text{for } T_A < T \leq T_B \quad (2.14)$$

$$S(T) = 2.5 \left(\frac{T_B}{T} \right)^{0.8} \quad \text{for } T_B < T \quad (2.15)$$

T_A and T_B are *Spectrum Characteristic Periods* as shown in the table below:

Table 2.5. Spectrum characteristic periods (T_A, T_B).

Local Soil Class as per TSC2007 Table 6.2	$T_A(\text{sec})$	$T_B(\text{sec})$
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

- Special Design Acceleration Spectra

TSC2007 Section 2.4.4 states that when required, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by Eq. 2.1 based on relevant characteristic periods specified in Table 2.4 [10].

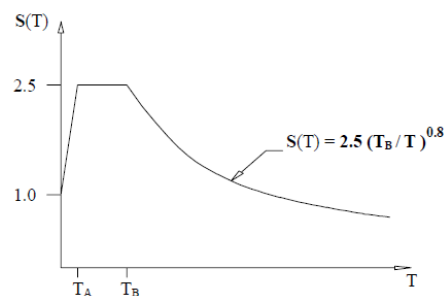


Figure 2.5. Elastic design spectrum.

- Equivalent Seismic Load Method

TSC2007 2.6.2 states that *Equivalent Seismic Load Method* can only be applied to buildings that satisfy the conditions described in Table 2.6 of TSC2007. This table is shown below:

Table 2.6. Buildings for which Equivalent Seismic Load Method can be applied.

Seismic Zone	Building Type	Total Height Limit
1,2	Buildings for which torsional irregularity coefficient satisfies $\eta_{bi} \leq 2.0$ at each storey	$H_N \leq 25\text{m}$
1,2	Buildings for which torsional irregularity coefficient satisfies $\eta_{bi} \leq 2.0$ at each storey and also B2 irregularity is NOT present	$H_N \leq 40\text{m}$
3,4	All buildings	$H_N \leq 40\text{m}$

The Total Equivalent Seismic Force (Base Shear) in relevant direction is governed by TSC2007 Eq. 2.4, and is shown below:

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10A_oIW \quad (2.16)$$

In the equation above;

W : Total weight of the building

R_a : Reduced Structural Behaviour Factor

$A(T_1)$: Spectral Acceleration Coefficient

A_o : Effective Ground Acceleration Coefficient

I : Building Importance Factor

Total weight of the building, W, shall be calculated according to TSC2007 Eq. 2.5;

$$W = \sum_{i=1}^N w_i \quad (2.17)$$

Where w_i , weight of i^{th} storey, is given by TSC 2007 Eq. 2.6;

$$w_i = g_i + nq_i \quad (2.18)$$

In Eq.2.10b;

g_i : Weight of i^{th} storey due to permanent loads

q_i : Weight of i^{th} storey due to live loads

n : Live load participation factor, as defined by TSC2007 Table 2.7

Table 2.7. Live load participation factor, n .

Purpose of Occupancy of Building	n
Warehouse, depot etc...	0.80
School, dormitory, sport facility, cinema, theater, concert hall, garage, restaurant, shop etc..	0.60
Residence, office, hotel, hospital etc...	0.30

According to TSC2007 2.7.2, *Total Equivalent Seismic Force*, V_t , shall be defined by the following expression:

$$V_t = \Delta F_N + \sum_{i=1}^N F_i \quad (2.19)$$

where

F_i : Seismic force acting on i^{th} storey

ΔF_N : Additional seismic force acting on n^{th} storey of the building

$$\Delta F_N = 0.0075 N V_t \quad (2.20)$$

The remaining part of the *Total Equivalent Seismic Force*, V_t , shall be distributed to the stories by the following expression:

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^N w_j H_j} \quad (2.21)$$

In the expression above, H_i and H_j are the heights of the i^{th} and j^{th} stories, respectively, as shown in Figure 2.6.

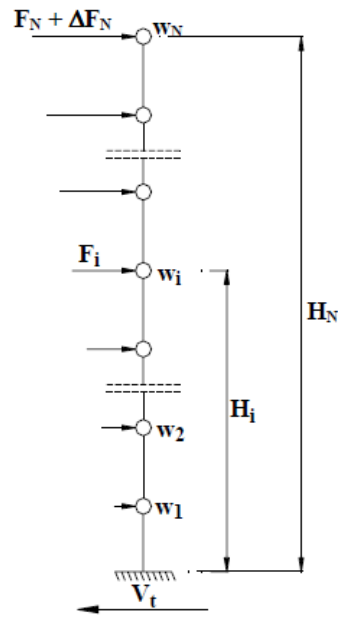


Figure 2.6. Distribution of lateral seismic forces [10].

TSC2007 Sec. 2.7.3.1 specifies that in buildings where floors behave as rigid horizontal diaphragms, two lateral displacement components and the rotation around the vertical axis shall be taken into account at each floor as independent static displacement components. At each floor, equivalent seismic loads determined in accordance with 2.7.2 shall be applied in the form of concentrated loads to the floor mass center as well as to shifted mass centers to account for the additional eccentricity effects. Shifted mass centers are the points obtained by shifting the actual mass center by +5% and -5% times the floor length in the perpendicular direction to the earthquake direction considered [10].

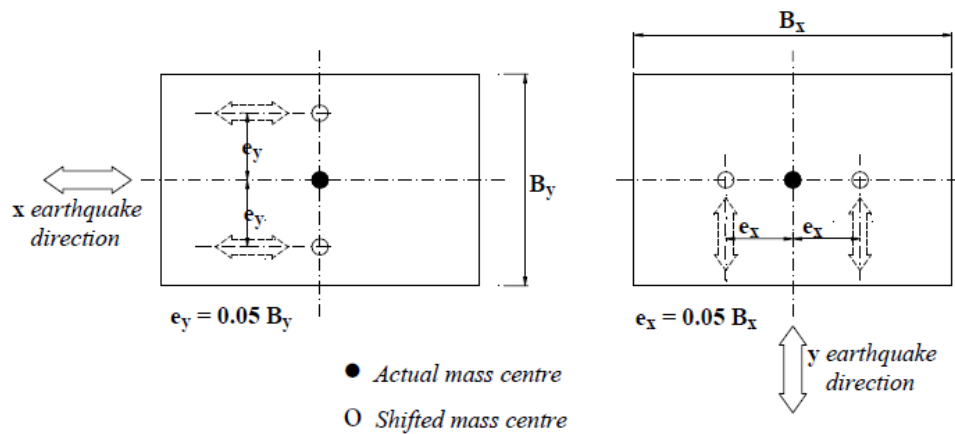


Figure 2.7. Actual and shifted mass centers [10].

TSC2007 2.7.3.3 states that in the case where type A1 irregularity defined in Table 2.1 exists at any i^{th} storey such that the condition $1.2 < \eta_{bi} < 2.0$ is satisfied, above defined 5% additional eccentricity applied to this floor shall be amplified by multiplying with coefficient D_i for both earthquake directions [10].

$$D_i = (\eta_{bi} / 1.2)^2 \quad (2.22)$$

Determination of *First Natural Vibration Period*, T_1 , of the structure is governed by TSC2007 Eq. 2.11, which is expressed below:

$$T_1 = 2\pi \left[\frac{\sum_{i=1}^N m_i d_{fi}^2}{\sum_{i=1}^N F_{fi} d_{fi}} \right]^{1/2} \quad (2.23)$$

In the above equation;

- m_i : Mass of i^{th} storey
- d_{fi} : Displacement of i^{th} storey subject to F_{fi}
- F_{fi} : Fictive load acting on i^{th} storey

Response quantities of structural elements with principal axes a and b nonparallel to earthquake directions is governed by TSC2007 Eq. 2.12, which is expressed below:

$$B_a = \pm B_{ax} \pm 0.30 B_{ay} \quad (2.24)$$

$$B_b = \pm B_{bx} \pm 0.30 B_{by} \quad (2.25)$$

$$B_a = \pm 0.30 B_{ax} \pm B_{ay} \quad (2.26)$$

$$B_b = \pm 0.30 B_{bx} \pm B_{by} \quad (2.27)$$

- Limitations on Relative Storey Drifts and Second-Order Effects

TSC2007 2.10.1.4 requires the following equation, TSC2007 Eq. 2.19, to be satisfied:

$$\frac{(\delta_i)_{\max}}{h_i} \leq 0.02 \quad (2.28)$$

In the above equation,

$(\delta_i)_{\max}$: Maximum effective relative storey drift

$$(\delta_i)_{\max} = R(d_i - d_{i-1})_{\max} \quad (2.29)$$

h_i : Height of i^{th} storey

TSC2007 2.10.2.2 specifies that Second Order Indicator Value, Θ_i , needs to satisfy the following expression:

$$\theta_i = \frac{(\Delta_i)_{\text{ort}} \sum_{j=i}^N w_j}{V_i h_i} \quad (2.30)$$

In the above equation;

V_i : Seismic shear force acting on i^{th} storey

$(\Delta_i)_{\text{ort}}$: Average reduced relative storey drift at i^{th} storey

- Clauses Regarding Ductility of Columns

Columns shall be considered to be of high ductility level provided they satisfy the requirements specified in TSC2007 Section 3.3 and listed below:

- For all columns, equation below shall be satisfied:

$$A_c \geq N_{dm} / (0.50 f_{ck}) \quad (2.31)$$

In Eq. 2.18;

A_c : Total cross-sectional area of column

N_{dm} : Maximum axial force due to combined effects of gravitational and seismic loads

f_{ck} : Characteristic compressive strength of concrete

- Minimum longitudinal reinforcement shall be 1% of cross-sectional area and maximum longitudinal reinforcement shall be 4% of cross-sectional area. At regions of lap splice, longitudinal reinforcement ratio shall not be larger than 6%.
- At bottom and top ends of each column, *confinement regions* shall be established. Length of confinement regions shall be the maximum of biggest column dimension, 1/6 of column clear length and 500mm. For cantilever columns, the confinement region's length shall not be smaller than twice the biggest column dimension.
- At confinement regions, transverse reinforcement with diameter smaller than 8mm shall not be used. The longitudinal distance between transverse reinforcement shall be the minimum of 1/3 of minimum column dimension and 100mm and shall not be greater than 50mm. Horizontal distance between arms of transverse reinforcement shall not be greater than 25 times the transverse reinforcement diameter.
- At confinement regions, if $N_D > 0.20A_c f_{ck}$, then maximum of the transverse reinforcement areas specified in the expressions below shall be provided:

$$A_{sh} \geq 0.30sb_k [(A_c / A_{ck}) - 1](f_{ck} / f_{ywk}) \quad (2.32)$$

$$A_{sh} \geq 0.075sb_k (f_{ck} / f_{ywk}) \quad (2.33)$$

In the expressions above;

A_{sh} : Area of transverse reinforcement required

s : Longitudinal distance between transverse reinforcement

b_k : Axial distance between transverse reinforcement that confines concrete

A_{ck} : Total area inside transverse reinforcement that confines concrete

f_{ywk} : Yield strength of transverse reinforcement

- If $N_D \leq 0.20A_c f_{ck}$, then 2/3 of the reinforcement required above should be provided.
- At structural systems that consist of only frames or a combination of walls and frames, summation of column ultimate strengths at each beam-column connection

shall be at least 20% larger than summation of beam ultimate strengths at the same connection.

$$(M_{ra} + M_{ru}) \geq 1.2(M_{ri} + M_{rj}) \quad (2.34)$$

Where

M_{ra}, M_{ru} : Ultimate strength of the column section below and above the connection, respectively

M_{ri}, M_{rj} : Ultimate strength of the beam section left and right of the connection, respectively

This requirement does not need to be satisfied for columns that cease at the relevant connection and for the case $N_D \leq 0.10A_c f_{ck}$.

- Shear analysis to determine required transverse reinforcement shall be based on V_e , obtained by the expression below:

$$V_e = (M_a + M_u) / l_n \quad (2.35)$$

M_a and M_u shall be determined as follows:

- If “*strong colum-weak beam criterion*” is met, summation of moment capacities of beams at the relevant connection, ΣM_p , shall be distributed to relevant column ends proportional to moments acting due to seismic forces. The distributed moments shall constitute M_a and M_u .
 - If “*strong column-weak beam criterion*” is not met, and for ends of columns that connect to the foundation, M_a and M_u shall be calculated as moment capacities M_r of relevant column ends. If more detailed analysis is not done, moment capacities can be assumed 1.4 times M_r .
- V_e shall in no case be greater than $V_r = 0.22A_w f_{cd}$ where A_w is the area of web section inside the column.

- Clauses Regarding Ductility of Beams

Beams shall be considered to be of high ductility level provided they satisfy the requirements specified in TSC2007 Section 3.4 and listed below:

- Web width of the beam shall be at least 250mm. Height of the beam shall not be less than 300mm and 3 times slab thickness and more than 3.5 times web width or ¼ of clear span length.
- Minimum tensile reinforcement ratio, ρ , shall satisfy the following expression:

$$\rho \geq 0.8 f_{ctd} / f_{yd} \quad (2.36)$$

Where

f_{ctd} : Design tensile strength of concrete

f_{yd} : Design yield strength of reinforcing steel

- Longitudinal reinforcement ratio shall not be greater than 2%.
- At beam supports, the regions with length equal to twice the height of beam depth shall be designed as “*confinement regions*”. Distance between the transverse reinforcement shall not be larger than 1/4 of beam height, 8 times the diameter of smallest longitudinal reinforcement and 150mm.
- Shear force to be used in determining required transverse reinforcement in beams, V_e , shall be obtained by the following expression:

$$V_e = V_{dy} \pm (M_{pi} + M_{pj}) / l_n \quad (2.37)$$

M_{pi} and M_{pj} are moment capacities of beam ends and can be assumed as 1.4 times ultimate moment capacities. V_{dy} is the shear force obtained by gravitational load analysis.

- V_e shall in no case be greater than $V_r = 0.22b_wdf_{cd}$ where b_w is the web width and d is the effective depth of the beam.
- Clauses Regarding Ductility of Beam-Column Connections

TSC2007 Section 3.5.1 specifies that “beam-column connections for frames with high ductility shall be classified to two parts as follows:

- If beams are connected to columns from four sides and the width of each beam is not less than 3/4 of column width, the connection shall be identified as a *sieged connection*.
- Connections that do not satisfy above conditions shall be identified as *unsieged connections*.

- According to TSC2007 Section 3.5.2.1, shear force at beam-column connections shall be obtained by the following expression:

$$V_e = 1.25f_{yk}(A_{s1} + A_{s2}) - V_{kol} \quad (2.38)$$

A_{s1} and A_{s2} are top and bottom longitudinal reinforcement of beams, respectively.

- Shear force V_e at connection shall satisfy the following conditions:
 - For sieged connections : $V_e \leq 0.60b_jhf_{cd}$ (2.39)
 - For unsieged connection : $V_e \leq 0.45b_jhf_{cd}$ (2.40)

- Minimum transverse reinforcement requirements for two kinds of connections specified in TSC2007 Section 3.5.2.3 and described below:

- For sieged connections, at least 40% of the transverse reinforcement determined for confinement region of the bottom column shall be used in connection region. Minimum diameter for transverse reinforcement shall be 8mm and the distance between transverse reinforcement shall not be larger than 150mm.

- (ii) For unsieged connections, at least 60% of the transverse reinforcement determined for confinement region of the bottom column shall be used in connection region. Minimum diameter for transverse reinforcement shall be 8mm and the distance between transverse reinforcement shall not be larger than 100mm.

2.6 Performance Analysis According to Turkish Seismic Code 2007

Requirements regarding performance analysis of existing buildings are specified in TSC2007 Section 7. Clauses relevant to this study shall be presented in this section.

- Levels of Information Gathered from Buildings

TSC2007 Section 7.2 describes in detail how to gather information from existing buildings and how the level of this information is reflected in the analysis process by *Information Level Factors*, tabulated in TSC2007 Table 7.1, as shown below:

Table 2.8. Information level factors as per TSC2007 Table 7.1.

Information Level	Information Level Factor
Limited	0.75
Intermediate	0.90
Advanced	1.00

In this study, pre-cast concrete buildings are analyzed and designed in accordance with TSC2007 requirements, and since this is an analytical study, the entirety of the information is known and the *Information Level Factor* is 1.0. Therefore, details about Information Levels shall not be presented here.

- Damage Limits and Damage Regions in Structural Members

TSC2007 Section 7.3 defines three limit states for sections of ductile members, namely *Minimum Damage Limit* (MN), *Safety Limit* (GV) and *Collapse Limit* (GÇ). These limit states, not applicable to brittle members, define the start of inelastic behaviour (MN),

the state at which the section can safely maintain its ultimate strength at inelastic behaviour (GV) and the state right before total collapse of the section (GÇ).

Section damage regions can be seen in the figure below. Members whose critical section does not reach MN are in Minimum Damage Region, those whose critical section does not reach GV are in Visible Damage Region, whose critical section stays between GV and GÇ are in Significant Damage Region and whose critical section exceeds GÇ are in Collapse Region.

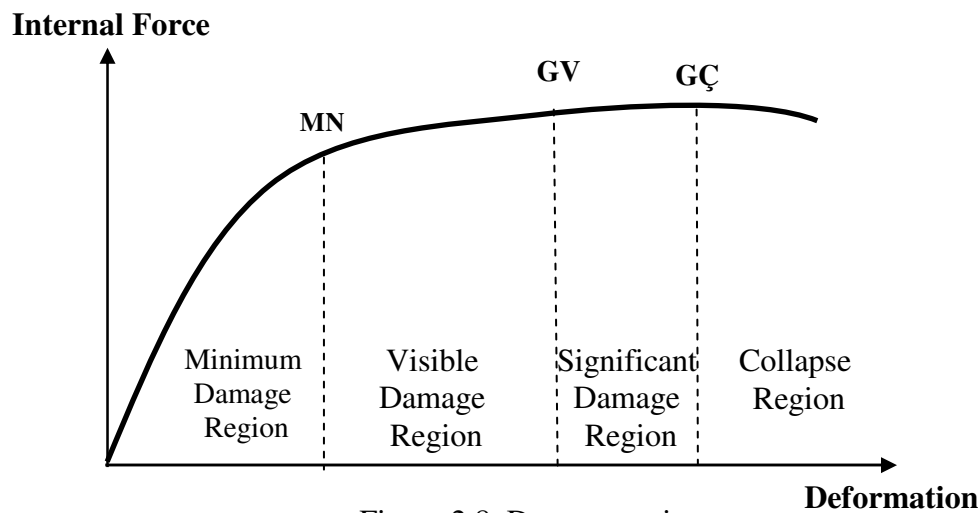


Figure 2.8. Damage regions.

- General Principals and Rules Regarding Seismic Analysis

General Principals and Rules Regarding Seismic Analysis, as specified in TSC2007 Section 7.4, are listed below:

- Linear Elastic and/or Nonlinear Inelastic analysis methods may be used. However, it should not be expected for these two methods to yield identical results.
- Seismic loads shall be defined using elastic (unreduced) acceleration spectrum given in 2.4, but relevant changes shall be made according to 7.8 for different probabilities of earthquakes to occur. Building Importance Factor, I , shall be taken equal to 1.0.

- Seismic performance of buildings shall be evaluated considering combined effects of gravitational and seismic loads. Live loads shall be determined in accordance with masses calculated for earthquake analysis.
- Seismic forces shall be separately applied in two orthogonal directions separately
- Mathematical model of the building shall represent the behaviour of the building under gravitational and seismic loads correctly
- Storey weights to be used in seismic analysis shall be determined according to 2.7.1.2, and story masses shall be defined in accordance with storey weights
- Buildings in which slabs act as rigid diaphragms, two horizontal and one rotational (about the vertical axis) degrees of freedom shall be defined. Storey degrees of freedom shall be defined at mass center of each storey and additional eccentricity shall not be applied
- Uncertainties for structural system of the building shall be reflected in the analysis using Information Level Factors defined in 7.2
- Requirements regarding interaction diagrams for sections subject to one or two-dimensional bending and axial forces are given below:
 - (i) In the analysis, *existing material strengths* shall be used for concrete and reinforcing steel.
 - (ii) Maximum compressive strain for concrete shall be taken equal to 0.003 and maximum strain for reinforcement steel shall be taken equal to 0.01
 - (iii) Interaction diagrams can be linearized conveniently
- In defining dimensions of reinforced concrete systems, connection regions can be assumed to be perfectly rigid regions

- *Effective bending rigidities $(EI)_e$* , shall be used for members subject to bending. Unless a more detailed analysis is performed, effective bending rigidities shall be determined by the following expressions

- (i) For beams : $(EI)_e = 0.40(EI)_o$
- (ii) For columns : $(EI)_e = 0.40(EI)_o$ if $N_D/(A_c f_{cm}) \leq 0.10$
: $(EI)_e = 0.80(EI)_o$ if $N_D/(A_c f_{cm}) \geq 0.40$

Where

$(EI)_o$: Uncracked section rigidity

A_c : Cross-sectional area

f_{cm} : Existing material strength for concrete

N_D : Axial force acting on the section determined by a preliminary gravitational load analysis using weights calculated in accordance with earthquake analysis

Linear interpolation can be made for intermediate levels of N_D .

- For reinforced concrete flanged sections, flange concrete and reinforcement inside the flange can be considered in ultimate strength analysis
- Determination of Building Performance by Linear Elastic Analysis Methods

Requirements regarding Building Performance by Linear Elastic Analysis Methods in addition to TSC2007 Section 2.7 and Section 2.8 are specified in TSC Section 7.5 and clauses relevant to this study are listed below:

- Equivalent Seismic Load Method shall be applied to buildings that satisfy the following conditions:
 - (i) Total height over basement is smaller than 25m
 - (ii) The building has no more than 8 stories
 - (iii) Torsional Irregularity Coefficient, η_{bi} , determined neglecting additional eccentricities, is smaller than 1.4
- Structural Behaviour Factor, R_a , shall be taken equal to 1.0

- Right side of equation 2.4 shall be multiplied by 1.0 for buildings that have one or two stories above the basement, and 0.80 for other buildings
- Determination of damage limits for reinforced concrete ductile members with linear elastic analysis methods shall be done using *demand/capacity ratios (r)*
- Reinforced concrete members are identified as ductile if their failure type is bending, brittle if their failure type is shear
 - (i) For columns, beams and walls to be identified as ductile, shear force V_e , determined in accordance with ultimate strengths of critical sections shall not exceed shear capacity V_r , determined in accordance with TS-500 considering existing material strengths. V_e shall be calculated in accordance with 3.3.7 for columns and 3.4.5 for beams. Ultimate strength moments shall be used to determine V_e . If the shear force calculated by seismic analysis taking $R_a=1$ is smaller than V_e , this shear force shall be considered.
 - (ii) Members that do not satisfy above conditions shall be considered brittle.
- Demand/capacity ratios for ductile column, beam and wall sections shall be calculated by dividing section moment determined by seismic analysis taking $R_a=1$ to section residual moment capacity. Relevant earthquake direction shall be considered in determining demand/capacity ratios.

Section residual moment capacity is the difference between the ultimate strength of the section and the moment calculated by gravitational analysis.

 - (i) Demand/capacity ratios for columns and walls shall be calculated in accordance with Appendix 7A.
 - (ii) Reinforced concrete columns that satisfy 3.3.4 and beams that satisfy 3.4.4 shall be identified as ‘confined’, and those who do not shall be identified as ‘unconfined’. ‘Confined’ members need to satisfy requirements of 3.2.8.
- Calculated demand/capacity ratios (r) shall be compared with limit values (r_s) and damage regions shall be determined. Linear interpolation may be used for

intermediate values. Limit values, defined in TSC2007 Tables 7.2 to 7.5 are shown below:

Table 2.9. Demand/capacity ratios (r_s) defining damage limits for reinforced concrete beams.

Ductile Beams			Damage Level		
$(\rho-\rho')/\rho_b$	Confinement	$V_e/(bw*_d*f_{ctm})$	MN	GV	GÇ
≤ 0.0	Yes	≤ 0.65	3	7	10
≤ 0.0	Yes	≥ 1.30	2.5	5	8
≥ 0.5	Yes	≤ 0.65	3	5	7
≥ 0.5	Yes	≥ 1.30	2.5	4	5
≤ 0.0	No	≤ 0.65	2.5	4	6
≤ 0.0	No	≥ 1.30	2	3	5
≥ 0.0	No	≤ 0.65	2	3	5
≥ 0.0	No	≥ 1.30	1.5	2.5	4

Table 2.10. Demand/capacity ratios (r_s) defining damage limits for reinforced concrete columns.

Ductile Columns			Damage Level		
$N_K/(Ac*f_{cm})$	Confinement	$V_e/(bw*_d*f_{ctm})$	MN	GV	GÇ
≤ 0.1	Yes	≤ 0.65	3	6	8
≤ 0.1	Yes	≥ 1.30	2.5	5	6
≥ 0.4 and ≤ 0.7	Yes	≤ 0.65	2	4	6
≥ 0.4 and ≤ 0.7	Yes	≥ 1.30	1.5	2.5	3.5
≤ 0.1	No	≤ 0.65	2	3.5	5
≤ 0.1	No	≥ 1.30	1.5	2.5	3.5
≥ 0.4 and ≤ 0.7	No	≤ 0.65	1.5	2	3
≥ 0.4 and ≤ 0.7	No	≥ 1.30	1	1.5	2
≥ 0.7	-	-	1	1	1

- At reinforced concrete beam-column connections, shear forces determined by Eq.3.11 acting on the connection should not be larger than shear strength defined in accordance with 3.5.2.2. However, at Eq.3.11, V_e , determined neglecting hardening, shall be used rather than V_{kol} . Existing material strengths shall be used in Eq 3.12 or 3.13. If connection shear force exceeds shear strength, the connection region shall be identified as brittle.
- For Linear Elastic Methods, at each earthquake directions, relative storey drifts at each storey level shall be compared with limit states given in the table below. If

state limits determined for storey drifts are worse than those determined before, these damage limits shall be considered.

Table 2.11. Damage limits for relative storey drifts.

Relative Storey Drift Ratio	Damage Limit		
	MN	GV	GÇ
Δ_{ji}/h_{ji}	0.01	0.03	0.04

- Determination of Building Performance by Nonlinear Inelastic Analysis Methods

TSC2007 Section 7.6.1 states that the objective of nonlinear inelastic performance methods for determining structural performance of existing buildings under seismic loading is determining plastic deformation demands related to ductile behaviour and internal force demands related to brittle behaviour for a given earthquake. These demands shall then be compared with deformation and force capacities defined in this section and performance of the sections and the building shall be evaluated.

- Procedure for Incremental Pushover Analysis to determine Building Performance

TSC2007 Section 7.6.3 specifies the following analysis procedure for Incremental Pushover Analysis to determine building performance:

- Prior to Incremental Pushover Analysis, a nonlinear static analysis considering loads in accordance with masses shall be performed. The results for this analysis shall constitute the initial phase of Incremental Pushover Analysis.
- If Incremental Equivalent Seismic Load Method is to be used, “modal capacity diagram” for the first mode, whose coordinates are defined as “modal displacement-modal acceleration”, shall be determined. Along with this diagram, modal displacement demand, determined considering relevant changes to account for different probabilities of occurrence of earthquake on the elastic spectrum defined in 2.4, shall be determined. At the last stage,

displacement, plastic deformation (plastic rotations) and internal force demands due to modal displacement demand shall be determined.

- (iii) Total curvature demands shall be calculated in accordance with 7.6.8 using plastic curvature demands determined for plastic (ductile) sections. Strain demands for concrete and reinforcement steel at reinforced concrete sections shall then be determined. These strain demands shall be compared with relevant strain capacities defined in 7.6.9 to evaluate section performance.
- (iv) For idealization of material nonlinearity, *lumped plastic behaviour model* shall be used. The *plastic deformation region*, to be named as *plastic hinge length* (L_p), shall be taken as equal to half the section height.

$$L_p = 0.5h \quad (2.26)$$

- (v) Plastic sections for beams and columns can be assumed to be at the edge of beam-column connections, in other words at the end of the clear length of members
- (vi) In determining internal force-plastic deformation relations, hardening effect (increment in plastic moment with increasing plastic rotation) may be neglected. If that is the case, plastic deformation vector shall be approximately orthogonal to the yield surface.

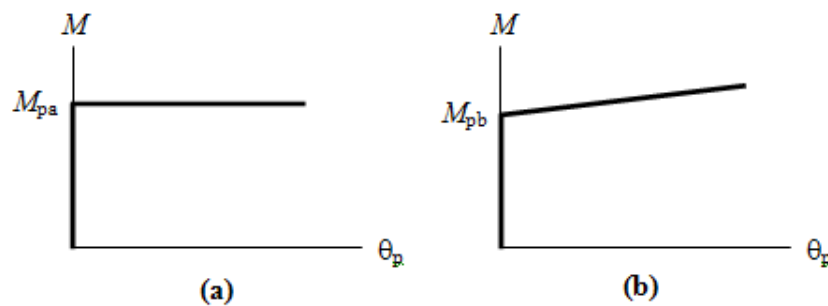


Figure 2.9. Plastic moment-plastic rotation relations (Fig 7.2 of [10]).

- Pushover Analysis with Incremental Equivalent Seismic Load Method

TSC2007 Section 7.6.5.1 states that the objective of Incremental Equivalent Seismic Load Method is to perform nonlinear pushover analysis using monotonically increasing seismic loads up to the earthquake demand level, proportional to first natural mode shape.

Plastic deformation and internal force increments shall be determined for each step following gravitational load analysis.

Requirements regarding Incremental Equivalent Seismic Load Method are specified in TSC2007 Section 7.6 and are listed below:

- Incremental Equivalent Seismic Load Method can only be applied to the buildings who have at most 8 stories excluding basement and those for which torsional irregularity constant η_{bi} , determined neglecting additional eccentricities, is smaller than 1.4 at each storey. In addition to those, ratio of the mass corresponding to the first vibration mode to the total building mass should be at least 0.70.
- During Incremental Pushover Analysis, it may be assumed that the distribution of equivalent seismic loads remains constant regardless of plastic hinge formations. The load distribution shall be defined to be proportional to the product of first natural vibration mode shape and the relevant mass.
- The Pushover Curve, whose coordinates are “*tip displacement-base shear force*”, shall be obtained by pushover analysis. The tip displacement is the displacement at the mass center of the top storey in relevant earthquake direction. The base shear force is the summation of equivalent earthquake loads at each step in relevant direction. *Modal capacity diagram*, whose coordinates are “*modal displacement-modal acceleration*”, shall be obtained by coordinate transformation by using the terms described below:

- (i) At i^{th} pushover step, modal acceleration $a_1^{(i)}$ shall be determined by the following expression:

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \quad (2.41)$$

- (ii) At i^{th} pushover step, modal displacement $d_1^{(i)}$ shall be determined by the following expression:

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\phi_{xN1} \Gamma_{x1}} \quad (2.42)$$

Modal participation factor Γ_{x1} , corresponding to first mode, shall be determined by the following expression:

$$\Gamma_{x1} = \frac{L_{x1}}{M_1} \quad (2.43)$$

In the above expression;

$$L_{x1} = m_i \phi_{xi1} \quad (2.44)$$

$$M_1 = m_i \phi_{xi1}^2 + m_i \Phi_{yi1}^2 + m_{\theta} \Phi_{\theta1}^2 \quad (2.45)$$

In Eq.2.29a and Eq.2.29b,

m_i : Mass of i^{th} storey

m_{θ} : For buildings at which slabs act as rigid diaphragms, mass moment of inertia about vertical axis

ϕ_{xi1} : For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey in x direction

ϕ_{yi1} : For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey in y direction

$\phi_{\theta1}$: For buildings at which slabs act as rigid diaphragms, the horizontal resultant of n^{th} mode shape at i^{th} storey about vertical axis

- Along with the modal capacity diagram, *modal displacement demand*, defined as maximum modal displacement corresponding to first mode, determined in accordance with the elastic behaviour spectrum defined in 2.4 shall be obtained. Modal displacement demand, $d_1^{(p)}$, is defined to be equal to *nonlinear spectral displacement*, S_{di1} :

$$d_{1(p)} = S_{di1} \quad (2.46)$$

(Clauses regarding determination of S_{di1} shall be described later in the section.)

- Tip displacement demand $u_{xN1}^{(p)}$ at x earthquake direction shall be determined by the following expression:

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} \quad (2.47)$$

All demand values (displacement, deformation or internal force demands) shall be determined using tip displacement demand.

- Determination of Strain Demands

TSC2007 7.6.8 specifies requirements regarding the determination of strain demands to be used in performance analysis. These requirements are listed below:

- *Plastic curvature demand*, ϕ_p , shall be determined using *plastic rotation demand* Θ_p by Eq. 2.32:

$$\Phi_p = \frac{\theta_p}{L_p} \quad (2.48)$$

- *Total curvature demand* at section, ϕ_t , shall be determined by adding plastic curvature demand ϕ_p to *equivalent yield curvature*, ϕ_y , which is obtained by moment-curvature analysis using relevant concrete and steel models

$$\Phi_t = \Phi_y + \Phi_p \quad (2.49)$$

- Strain Capacities for Reinforced Concrete Sections

TSC2007 Section 7.6.9 specifies that “strain demands for concrete and steel shall be compared with the strain capacities defined below and performance levels shall be determined.”

- Minimum Damage Limit (MN)

$$(\epsilon_{cu})_{MN} = 0.0035 ; (\epsilon_s)_{MN} = 0.010$$

- Safety Limit (GV)

$$(\epsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s / \rho_{sm}) \leq 0.0135 ; (\epsilon_s)_{GV} = 0.040$$

- Collapse Limit (GC)

$$(\epsilon_{cg})_{GC} = 0.004 + 0.014(\rho_s / \rho_{sm}) \leq 0.018 ; (\epsilon_s)_{GC} = 0.060$$

- Shear Capacities of Reinforced Concrete Members

Shear capacities for all members, except for beam-column connection regions, shall be calculated according to TS-500 using existing material strengths. Members for which shear capacity is smaller than shear demand shall be identified as brittle.

Shear demand at beam-column connections should not exceed shear capacity of the region. However, V_{kol} in Eq 3.11 shall be replaced with shear demand due to earthquake and shear capacities shall be calculated using existing material strengths. Connections for which shear capacity is smaller than shear demand shall be identified as brittle.

- Determination of Building Performance

TSC2007 Section 7.7 specifies the procedure for determining performance level:

- (i) *Immediate Usage Performance Level:* At each storey and for each earthquake direction, at most 10% of beams shall be in *Visible Damage Region*, but all of the other members shall be in *Minimum Damage Region*.
- (ii) *Life Safety Performance Level:* At each storey and for each earthquake direction, at most 30% of beams except for secondary ones shall be in *Significant Damage Level*. In addition, contribution of columns that are in *Significant Damage Level* to the shear force that is resisted by the columns at the relevant storey should be less than 20%. At top storey, this limit can be increased to 40%. All remaining members shall be in *Minimum Damage Region* or *Visible Damage Region*. However, shear force resisted by columns whose top and bottom sections both exceed *Minimum Damage Level* to the shear force resisted by all columns at the relevant storey shall not be greater than 30%. (For linear elastic analysis, columns whose top and bottom sections both satisfy Eq. 3.3 do not need to satisfy this latter condition)
- (iii) *Collapse Prevention Performance Level:* It should be considered that brittle members are in *Collapse Region*. At each storey and for each earthquake direction, at most 20% of beams except for secondary ones shall be in *Collapse Region*. All remaining members shall be in *Minimum Damage Region*, *Visible Damage Region* or *Significant Damage Region*. However, shear force resisted by columns whose top and bottom sections both exceed *Minimum Damage*

Level to the shear force resisted by all columns at the relevant storey shall not be greater than 30%. (For linear elastic analysis, columns whose top and bottom sections both satisfy Eq. 3.3 do not need to satisfy this latter condition) The occupancy of the building is not safe.

- (iv) *Collapse State*: If the building does not satisfy *Prior-to-Collapse Performance Level*, then it is in *Collapse State*. The occupancy of the building is not safe.

- Target Performance Levels for Buildings

TSC 2007 Section 7.8.1 states that the acceleration spectrum defined in 2.4 for analysis of new structures is based on 10% probability of occurrence of earthquake in 50 years. Ordinates for the earthquake that has a 50% probability of occurrence in 50 years shall be taken as half of that defined in 2.4. Ordinates for the earthquake that has a 2% probability of occurrence in 50 years shall be taken as 1.5 times that defined in 2.4.

Table 2.12. Target performance levels for buildings.

Purpose of Occupancy and Type of Building	Probability of exceedance		
	50% in 50 years	10% in 50 years	2% in 50 years
The buildings to be utilized immediately after the earthquake: Hospitals, health facilities, fire fighting buildings, communication and power facilities, transportation stations, governorate, county and municipality administration buildings, first aid and emergency planning stations	-	HK	CG
Intensively and long-term occupied Buildings: Schools, dormitories, boarding houses, military barracks, prisons, museums, etc.	-	HK	CG
Intensively and short-term occupied Buildings: Cinema, theatre, concert saloons, cultural centers, sport facilities.	HK	CG	-
Buildings containing hazardous materials: Buildings containing or storing toxic, explosive and flammable materials, etc.	-	HK	GÖ
Other buildings: Buildings after than above defined buildings (Residential and office buildings, hotels, industrial structures, etc.)	-	CG	-

- Determination of Demand/Capacity Ratios for Columns

TSC2007 Appendix 7A defines the procedure for determining demand/capacity ratios for reinforced concrete columns and walls.

A linearized moment-curvature diagram for a column member can be seen in the figure below. Point D defines moment M_D and axial force N_D due to gravitational loads. Horizontal and vertical projections of the line that starts from point D and goes out of the interaction diagram defines M_E - N_E couple that is determined by earthquake analysis by taking $R=1$. Point K, defined as the point where the second line intersects the interaction diagram, represent the moment capacity M_K and corresponding axial force N_K .

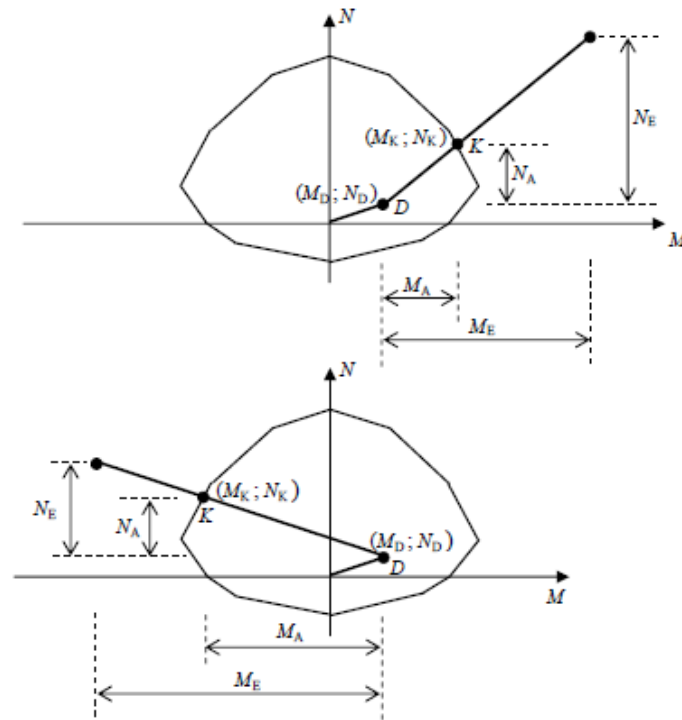


Figure 2.10. Moment-interaction diagrams and demand/capacity ratios.

Residual moment capacity M_A and corresponding axial force N_A are defined by the following expressions:

$$M_A = M_K - M_D \quad (2.50)$$

$$N_A = N_K - N_D \quad (2.51)$$

Demand/capacity ratio for the member is defined by the following expression:

$$r = \frac{M_E}{M_A} = \frac{N_E}{N_A} \leq r_s \quad (2.52)$$

- Stress-Strain Relations for Concrete and Reinforcing Steel

Stress-strain relations for concrete and reinforcing steel to be used in nonlinear inelastic performance analysis is given by TSC2007 Appendix 7B and described below:

- Confined-Unconfined Concrete Model (Mander Model)

Concrete stress-strain relationship model commonly known as the “Mander Model” [11] is specified to be convenient by TSC2007. Stress-strain diagram and the expressions defining stress-strain relations are given below.

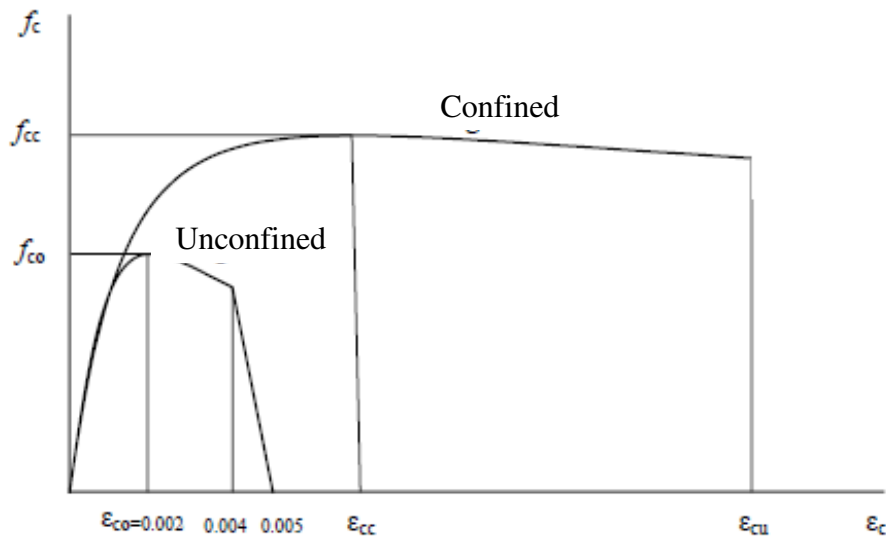


Figure 2.11. Stress-strain diagram for Mander Model.

Compressive stress in confined concrete f_c is defined by the following expression:

$$f_c = \frac{f_{cc} x^r}{r - 1 + x^r} \quad (2.53)$$

In Eq.2.37;

f_{cc} : Confined concrete strength

x : Normalized concrete strain, given by the following expression:

$$x = \frac{\epsilon_c}{\epsilon_{cc}}; \epsilon_{cc} = \epsilon_{co} [1 + 5(\lambda_c - 1)]; \epsilon_{co} = 0.002 \quad (2.54)$$

r : A variable given by the expression below:

$$r = \frac{E_c}{E_c - E_{sec}}; E_c = 5000\sqrt{f_{co}}; E_{sec} = \frac{f_{cc}}{\epsilon_{cc}} \quad (2.55)$$

Confined concrete strength f_{cc} is defined by the following expression:

$$f_{cc} = \lambda_c f_{co} \quad (2.56)$$

Where;

f_{co} : Compressive strength of unconfined concrete

λ_c : A variable given by the following expression:

$$\lambda_c = 2.254 \sqrt{1 + 7.94 \frac{f_e}{f_{co}}} - 2 \frac{f_e}{f_{co}} - 1.254 \quad (2.57)$$

In the above expression:

f_e : Effective confining stress, defined as the average value for the two orthogonal directions defined below:

$$f_{ex} = k_e \rho_x f_{yw}; f_{ey} = k_e \rho_y f_{yw} \quad (2.58)$$

Where

k_e : Confining effectiveness factor, given by the following expression:

$$k_e = \left(1 - \frac{\sum a_i^2}{6b_o h_o} \right) \left(1 - \frac{s}{2b_o} \right) \left(1 - \frac{s}{2h_o} \right) \left(1 - \frac{A_s}{b_o h_o} \right)^{-1} \quad (2.59)$$

$\sum a_i^2$: Sum of square of clear distance between longitudinal rebars

b_o, h_o : Distance between axes of stirrups confining concrete

s : Stirrup distance

A_s : Longitudinal rebar area

ρ_x : Volumetric ratio of transverse reinforcement in x-direction

ρ_y : Volumetric ratio of transverse reinforcement in y-direction

Maximum concrete strain for confined concrete, ϵ_{cu} , is given by the expression below:

$$\epsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{yw} \epsilon_{su}}{f_{cc}} \quad (2.60)$$

In the expression above:

ρ_s : Total volumetric ratio of transverse reinforcement ($\rho_x + \rho_y$)

- Reinforcing Steel Model

TSC Appendix 7.B.2 specifies following stress-strain relations for reinforcing steel:

$$f_s = E_s \epsilon_s \quad (\epsilon_s \leq \epsilon_{sy}) \quad (2.61)$$

$$f_s = f_{sy} \quad (\epsilon_{sy} < \epsilon_s \leq \epsilon_{sh}) \quad (2.62)$$

$$f_s = f_{su} - (f_{su} - f_{sy}) \frac{(\epsilon_{su} - \epsilon_s)^2}{(\epsilon_{su} - \epsilon_{sh})^2} \quad (\epsilon_{sh} < \epsilon_s \leq \epsilon_{su}) \quad (2.63)$$

Table 2.13. Strength-strain values for reinforcing steel.

Grade	f_{sy} (MPa)	ϵ_{sy}	ϵ_{sh}	ϵ_{su}	f_{su} (MPa)
S220	220	0.0011	0.011	0.16	275
S420	420	0.0021	0.008	0.10	550

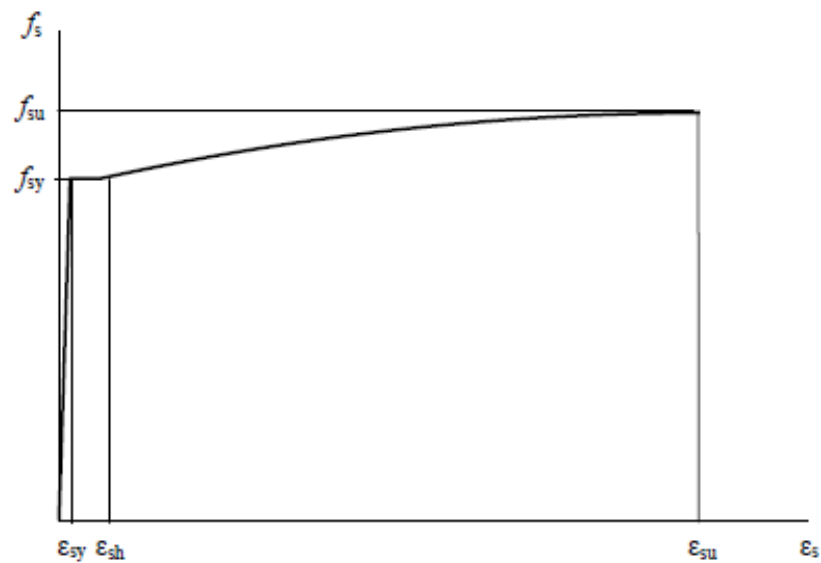


Figure 2.12. Stress-strain diagram for reinforcing steel.

- Determination of Nonlinear Spectral Displacement

TSC2007 Appendix 7C specifies the procedure for determining nonlinear spectral displacement to be used in nonlinear performance analysis.

Nonlinear spectral displacement corresponding to first natural mode S_{d1} is given by the following expression:

$$S_{d1} = C_{R1} S_{de1}; S_{de1} = \frac{S_{ae1}}{(w_1^{(1)})^2} \quad (2.64)$$

Spectral Displacement Ratio C_{R1} is determined based on $T_1^{(1)}$, natural vibration period corresponding to first vibration mode at first step:

- If $T_1^{(1)} \geq T_B$, C_{R1} shall be taken equal to unity ($C_{R1}=1$)

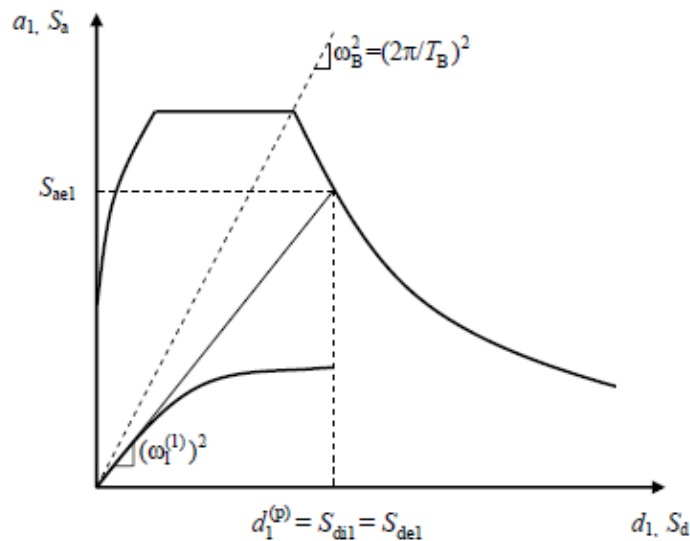


Figure 2.13. Spectral displacement-spectral acceleration and modal capacity diagram ($C_{R1}=1$).

- If $T_1^{(1)} < T_B$, C_{R1} shall be determined as follows:
 - (i) Modal capacity diagram determined by pushover analysis shall be transformed into a bi-linear diagram as shown in the figure below. The slope of starting line of this diagram shall be equal to $(\omega_1^{(1)})^2$

- (ii) At the first step of consecutive approach, the coordinates for *equivalent yield point* shall be determined by equal area rule by taking $C_{R1}=1$. Based on a_{y1}^0 shown in the figure below, C_{R1} is then determined by the following expression:

$$C_{R1} = \frac{1 + (R_{y1} - 1)T_B / T_1^{(1)}}{R_{y1}} \geq 1 \quad (2.65)$$

Where R_{y1} is the strength reduction factor and given by:

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \quad (2.66)$$

- (iii) Equal area rule shall be used to recalculate a_{y1} , R_{y1} and C_{R1} until consecutive values are close enough.

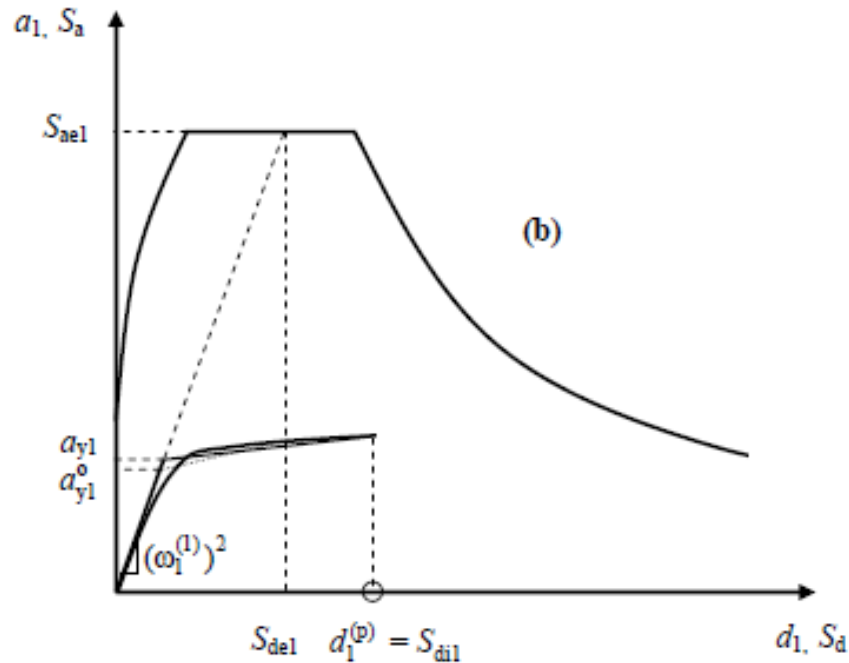


Figure 2.14. Spectral displacement-spectral Acceleration and modal capacity diagram ($C_{R1} \neq 1$).

3. SEISMIC ANALYSIS AND DESIGN OF BUILDINGS

Two different structures with pre-cast members, Building 1 and Building 2, are analyzed and designed in this study. Two additional structures, namely Building 1A and Building 2A, are also analyzed and designed. Building 1A and Building 2A are analyzed using *cracked section rigidities* as per TSC2007 Section 7.4.13 where Building 1 and Building 2 are analyzed using *uncracked section rigidities* as per TSC2007 Section 3.2.3.

Seismic analysis and design is performed by SAP2000 [12] analysis software.

All structures have the same 10m x 10m axial bay and span dimensions, as shown in Figure 3.1. Storey height for Building 1 and Building 1A is 10m where storey height for Building 2 and Building 2A is 7m, as shown in Figure 3.2 and Figure 3.3.

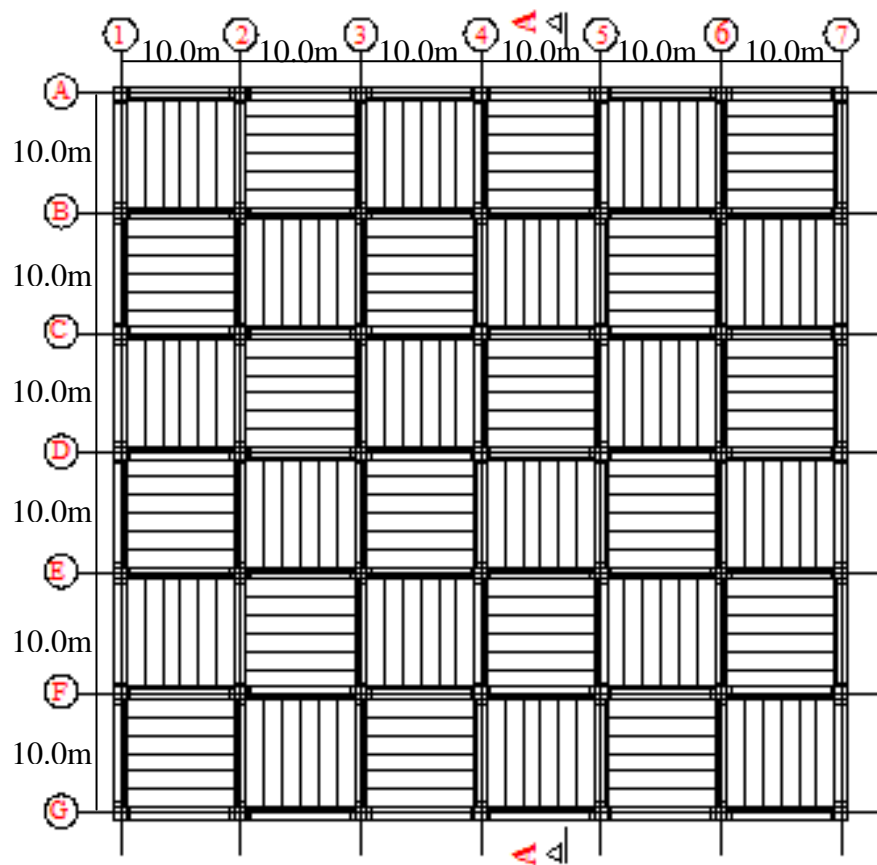


Figure 3.1. Plan view of buildings.

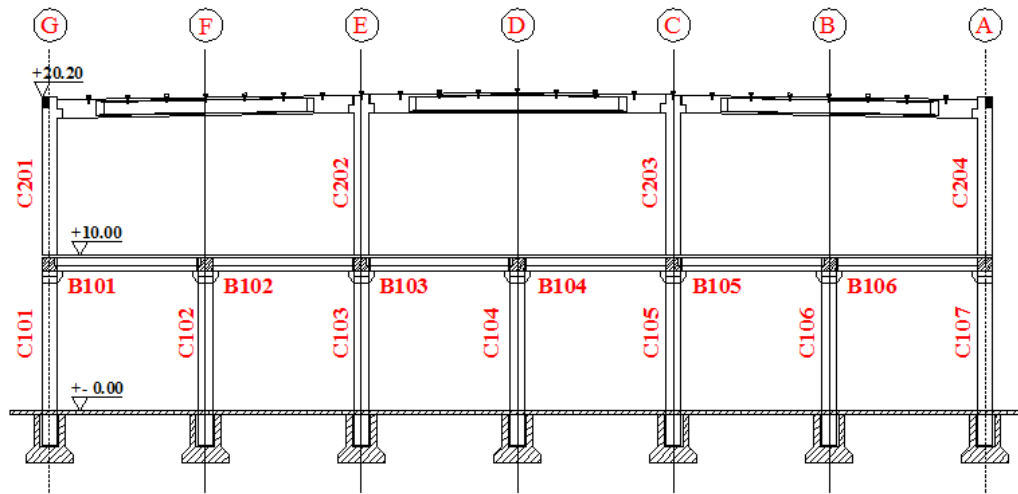


Figure 3.2. Section A-A for Building 1 and Building 1A (h=10m).

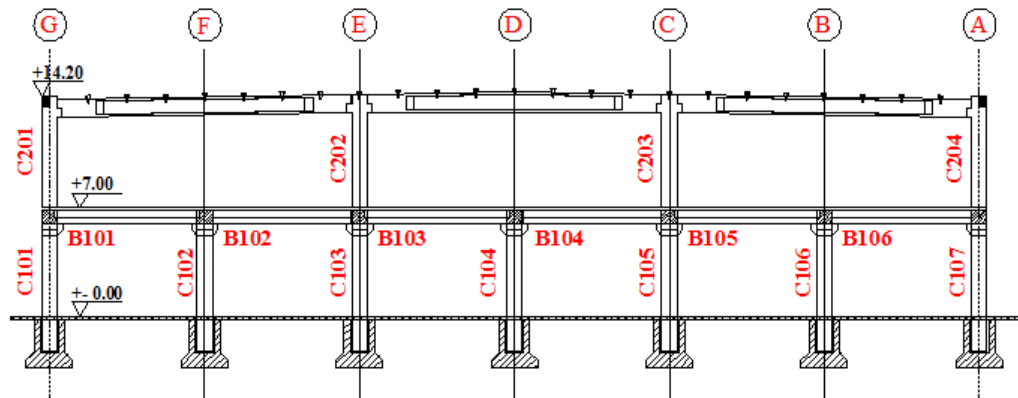


Figure 3.3. Section A-A for Building 2 and Building 2A (h=7m).

In this study, analysis and design of individual pre-cast members for which analysis is not dependent on earthquake analysis (frame behaviour), namely purlins, roof girders and storey slabs, are done by specially developed software and shall not be presented here.

Analysis parameters used in the study are presented in Section 3.1, building irregularity and storey drift analyses are presented in Section 3.2, column design is presented in Section 3.3, beam and beam-column connection design is presented in Section 3.4.

3.1 Analysis Parameters

Analysis parameters used in this study are listed below:

- Material
 - Concrete
 - Pre-cast Columns : C30 ($f_{ck}=30$ MPa, $f_{ctk}=1.9$ MPa, $E_c=32,000$ MPa)
 - Pre-stressed Beams : C40 ($f_{ck}=40$ MPa, $f_{ctk}=2.2$ MPa, $E_c=34,000$ MPa)
 - In-situ Concrete : C25 ($f_{ck}=25$ MPa, $f_{ctk}=1.8$ MPa, $E_c=30,000$ MPa)
 - Reinforcement
 - All RC Members : S420a ($f_{yk}=420$ MPa, $f_{su}=500$ MPa)
 - Pre-stressing Strands
 - All Pre-stressed Members : 1/2'' 270K ($A_{ps}=0.987\text{cm}^2$, $f_{pk}=1860\text{MPa}$)
- Loads
 - Roof Loading
 - Coating : 0.50kN/m^2
 - Snow : 0.75kN/m^2
 - Storey Loading
 - Live Loading : 25.00kN/m^2
- Earthquake Parameters
 - Earthquake Zone : 1.
 - Effective ground acceleration coefficient, A_0 : 0.40
 - Building Importance Factor, I : 1.0 (Industrial Building)
 - Soil Class : Z3
 - Live Load Reduction Factor, n
 - : 0.30 (Roof/Snow)
 - : 0.80 (Storey/Warehouse)
- Load Combinations
 - 1.0DL + 1.0LL
 - 1.4DL + 1.6LL

$$1.0DL + 1.0LL \pm 1.0EX \pm 0.3EY$$

$$1.0DL + 1.0LL \pm 0.3EX \pm 1.0EY$$

$$0.9DL \pm 1.0EX \pm 0.3EY$$

$$0.9DL \pm 0.3EX \pm 1.0EY$$

Where

DL : Dead Loads

LL : Live Loads

EX : Seismic Loads in x-direction

EY : Seismic Loads in y-direction

It should be noted here that in calculating total building weight, 0.80 of the live loading, apart from the live load participation factor n , was considered to account for the fact that not the total area of the floor shall be loaded at the instant of an earthquake.

3.2 Building Irregularities and Storey Drifts

Building irregularities and storey drifts determined by seismic analyses are tabulated below. Note that none of the buildings have A1, B1 and B2 irregularities. Column dimension requirements are governed by storey drift limitation (0.02) of TSC2007. Column dimensions are 100/100 for Building 1, 160/160 for Building 1A, 85/85 for Building 2 and 130/130 for Building 2A. Storey drifts are all at the 0.02 limit. *Second-order effect indicator*, Θ , for all buildings is much smaller than the limit value, 0.12. Notations are described in Section 2.5.

Table 3.1. Building irregularities, storey drifts and second-order indicator checks.

Building	Column Size(cm)	A1	B1	B2	Storey Drifts		P- Δ	
		η_{b1}	η_{c1}	η_{k1}	$(\Delta_1)/h_1$	$(\Delta_2)/h_2$	Θ_1	Θ_2
B1	100/100	1.08	1.75	1.61	0.011	0.020	0.017	0.012
B1A	160/160	1.06	1.75	1.78	0.010	0.020	0.017	0.014
B2	85/85	1.13	1.75	1.56	0.011	0.019	0.012	0.011
B2A	130/130	1.07	1.75	1.75	0.010	0.020	0.010	0.009

3.3 Column Design

Column design details, such as column sizing, required longitudinal and transverse reinforcement areas and provided longitudinal and transverse reinforcement areas are presented in this section. Column details for Building 1, Building 1A, Building 2 and Building 2A are tabulated in Table 3.2, Table 3.3, Table 3.4 and Table 3.5, respectively.

It should be noted that longitudinal reinforcement area much more than that needed for internal forces is provided for middle columns that go up to the roof, namely C103 and C105, to satisfy strong column-weak beam criterion specified in TSC2007 Section 3.3.5 and described in Section 2.5 of this study.

Required longitudinal reinforcement ratio for all columns of Building 1A and Building 2A is minimum required ratio (1%) as specified in TSC2007 Section 3.3.2.1. This is due to the fact that in order to satisfy relative storey drift requirement for these columns, which are analyzed using cracked section rigidities, very large cross sections are needed.

Transverse reinforcement required is governed by TSC2007 Eq. 3.1 (Eq 2.19b in this study), which is also described in Section 2.5.

In the tables below, the notations –BOT and –TOP represent bottom and top sections of corresponding members, respectively. For instance, C101-TOP represents top section of column C101. Other notations for the tables are listed below:

- $A_{s,req}$: Required longitudinal reinforcement area (cm^2)
- ϕ_{prov} : Provided rebars
- $A_{s,prov}$: Provided longitudinal reinforcement area (cm^2)
- $(A_s/s)_{req}$: Required transverse reinforcement area / distance (cm)
- $(A_s/s)_{prov}$: Provided transverse reinforcement area / distance (cm)

Table 3.2. Building 1 column design details.

Column	Longitudinal Reinforcement			Transverse Reinforcement		
	$A_{s,req}$	ϕ_{prov}	$A_{s,prov}$	$(A_s/s)_{req}$	ϕ_{prov}	$(A_s/s)_{prov}$
C101-BOT	155	12 ϕ 28+16 ϕ 26	159	0.329	6 ϕ 10/12.5	0.375
C101-TOP	127	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C201-BOT	175	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C102-BOT	169	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C102-TOP	162	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C103-BOT	121	24 ϕ 26	127	0.329	6 ϕ 10/12.5	0.375
C103-TOP	100	16 ϕ 32+24 ϕ 26	256	0.329	6 ϕ 10/12.5	0.375
C202-BOT	235	16 ϕ 32+24 ϕ 26	256	0.329	6 ϕ 10/12.5	0.375
C104-BOT	167	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C104-TOP	158	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C105-BOT	115	24 ϕ 26	127	0.329	6 ϕ 10/12.5	0.375
C105-TOP	100	16 ϕ 32+24 ϕ 26	256	0.329	6 ϕ 10/12.5	0.375
C203-BOT	230	16 ϕ 32+24 ϕ 26	256	0.329	6 ϕ 10/12.5	0.375
C106-BOT	169	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C106-TOP	162	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C107-BOT	157	12 ϕ 28+16 ϕ 26	159	0.329	6 ϕ 10/12.5	0.375
C107-TOP	119	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375
C204-BOT	174	12 ϕ 32+16 ϕ 26	181	0.329	6 ϕ 10/12.5	0.375

Table 3.3. Building 1A column design details.

Column	Longitudinal Reinforcement			Transverse Reinforcement		
	$A_{s,req}$	ϕ_{prov}	$A_{s,prov}$	$(A_s/s)_{req}$	ϕ_{prov}	$(A_s/s)_{prov}$
C101-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C101-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C201-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C102-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C102-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C103-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C103-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C202-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C104-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C104-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C105-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C105-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C203-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C106-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C106-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C107-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C107-TOP	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542
C204-BOT	256	12 ϕ 30+28 ϕ 28	257	0.543	6 ϕ 12/12.5	0.542

Table 3.4. Building 2 column design details.

Column	Longitudinal Reinforcement			Transverse Reinforcement		
	$A_{s,req}$	ϕ_{prov}	$A_{s,prov}$	$(A_s/s)_{req}$	ϕ_{prov}	$(A_s/s)_{prov}$
C101-BOT	132	12 ϕ 26+16 ϕ 24	136	0.418	4 ϕ 12/10	0.452
C101-TOP	116	24 ϕ 26+4 ϕ 32	159	0.418	4 ϕ 12/10	0.452
C201-BOT	155	24 ϕ 26+4 ϕ 32	159	0.418	4 ϕ 12/10	0.452
C102-BOT	154	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C102-TOP	138	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C103-BOT	110	24 ϕ 24	109	0.418	4 ϕ 12/10	0.452
C103-TOP	72	28 ϕ 34+4 ϕ 32	286	0.418	4 ϕ 12/10	0.452
C202-BOT	192	28 ϕ 34+4 ϕ 32	286	0.418	4 ϕ 12/10	0.452
C104-BOT	144	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C104-TOP	131	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C105-BOT	109	24 ϕ 24	109	0.418	4 ϕ 12/10	0.452
C105-TOP	72	28 ϕ 34+4 ϕ 32	286	0.418	4 ϕ 12/10	0.452
C203-BOT	192	28 ϕ 34+4 ϕ 32	286	0.418	4 ϕ 12/10	0.452
C106-BOT	154	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C106-TOP	138	12 ϕ 28+16 ϕ 26	159	0.418	4 ϕ 12/10	0.452
C107-BOT	122	12 ϕ 26+16 ϕ 24	136	0.418	4 ϕ 12/10	0.452
C107-TOP	118	24 ϕ 26+4 ϕ 32	159	0.418	4 ϕ 12/10	0.452
C204-BOT	153	24 ϕ 26+4 ϕ 32	159	0.418	4 ϕ 12/10	0.452

Table 3.5. Building 2A column design details.

Column	Longitudinal Reinforcement			Transverse Reinforcement		
	$A_{s,req}$	ϕ_{prov}	$A_{s,prov}$	$(A_s/s)_{req}$	ϕ_{prov}	$(A_s/s)_{prov}$
C101-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C101-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C201-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C102-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C102-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C103-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C103-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C202-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C104-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C104-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C105-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C105-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C203-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C106-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C106-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C107-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C107-TOP	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437
C204-BOT	169	12 ϕ 32+16 ϕ 24	169	0.436	7 ϕ 10/12.5	0.437

3.4 Beam and Beam-Column Connection Design

Pre-cast beam members of the structures are designed according to gravitational and seismic loads. Beam design and beam-column connection design are both presented here since longitudinal reinforcement is governed by ‘adequate force transfer’ specified in TSC2007 3.12.2.2 and described in Section 1.2.(iii) of this study.

Section of the typical beam can be seen in the figure below. Total (composite) height of the section is 100cm and web width of the section is 80cm. Note that the web corbel of the beam is necessary to constitute support for pre-cast slabs.

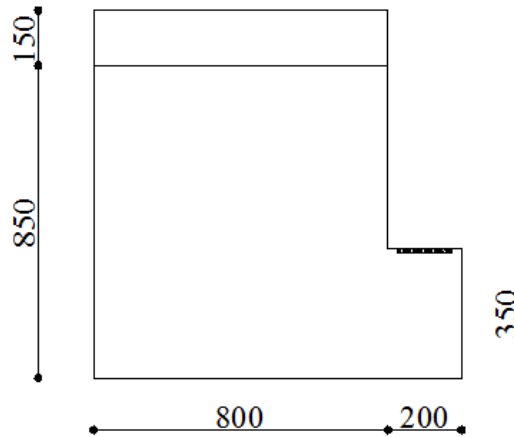


Figure 3.4. Sectional dimensions of the typical composite beam.

Longitudinal reinforcement is welded to the embedded plate at the beam end for transfer of forces due to positive moment, as shown in the figure below. Seismic forces are multiplied by two and welding is designed accordingly. Required longitudinal reinforcement is generally governed by weld design, in other words to ensure adequate welding length.

Ultimate strength analysis of pre-stressed members is done according to Turkish Pre-stressed Concrete Code TS-3233 principles which are described in detail in Section 2.4 of this study.

It should be noted here that usual practice is designing the beam end for the worst case, meaning for the maximum force, and applying the detail to all connections. To be consistent with practice, same approach is used in this study. Therefore, results for only one beam for each building are presented in this section.

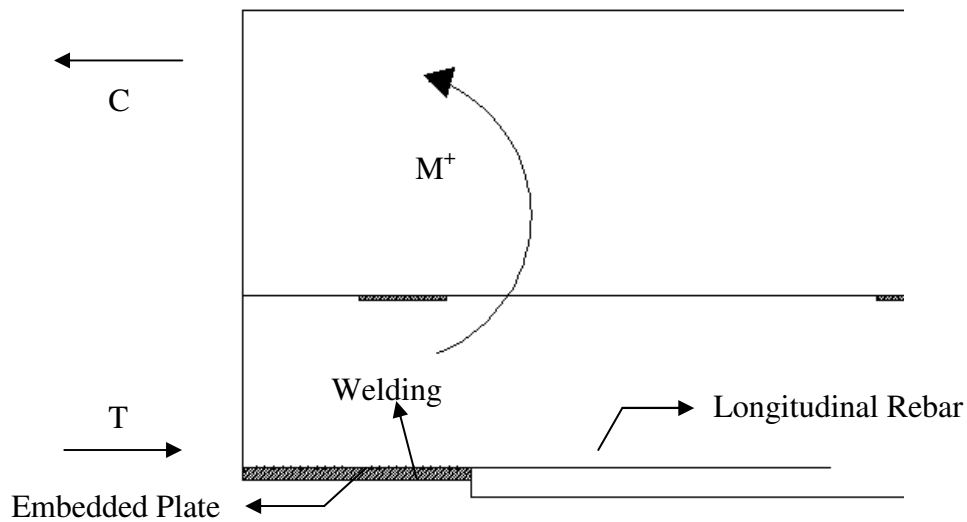


Figure 3.5. Beam end force transfer mechanism.

Longitudinal and transverse reinforcement design for beams of all buildings are summarized in Table 3.6.

Table 3.6. Design details for beams.

Member		Bottom Reinforcement	Top Reinforcement	Transverse Reinforcement
Building1	Support	10x0.5''+5 ϕ 26+3 ϕ 32	10 ϕ 32	4 ϕ 12/10
	Span	10x0.5''+5 ϕ 26	9 ϕ 26	4 ϕ 10/20
Building1A	Support	10x0.5''+5 ϕ 26	7 ϕ 32	4 ϕ 12/10
	Span	10x0.5''+5 ϕ 26	9 ϕ 26	4 ϕ 10/20
Building2	Support	10x0.5''+5 ϕ 26+2 ϕ 32	10 ϕ 32	4 ϕ 12/10
	Span	10x0.5''+5 ϕ 26	9 ϕ 26	4 ϕ 10/20
Building2A	Support	10x0.5''+5 ϕ 26	7 ϕ 32	4 ϕ 12/10
	Span	10x0.5''+5 ϕ 26	9 ϕ 26	4 ϕ 10/20

4. PERFORMANCE ANALYSIS OF STRUCTURES WITH LINEAR ELASTIC METHOD

This chapter of the study describes in detail the methodology of performance analysis of the four structures using linear elastic methods. Since the specifications of TSC2007 for performance analysis with linear analysis methods are described in Section 2.6 of this study, mainly the numerical parameters and results, with relevant references to Section 2.6, are presented in this chapter.

It is crucial to note that although initial analysis and design of the four buildings are performed using 3D mathematical models, accounting for additional eccentricities and effects of seismic forces in orthogonal directions simultaneously, planar analyses are performed in performance evaluation for simplicity. Undoubtedly, this fact affects interpretation of the results obtained and needs to be paid attention.

2D mathematical models representing the frames are prepared and structural analyses which will be the base for performance evaluation are performed by utilizing SAP2000 [12] structural analysis software.

Analysis parameters used in linear elastic performance analysis of the buildings are identical to those used for initial analysis and design phase and presented in Section 3.1. However, relevant changes for analysis parameters, such as taking $R=1$ and neglecting the effects of seismic forces in the perpendicular direction, are ofcourse made.

4.1 Performance Analysis of Building 1 with Linear Elastic Method

Performance evaluation of Building 1 with Linear Elastic Methods is presented in this section. 2D frame model of the building, shown in Fig. 4.1, is prepared in SAP2000 software.

4.1.1 Mathematical Model for Building 1

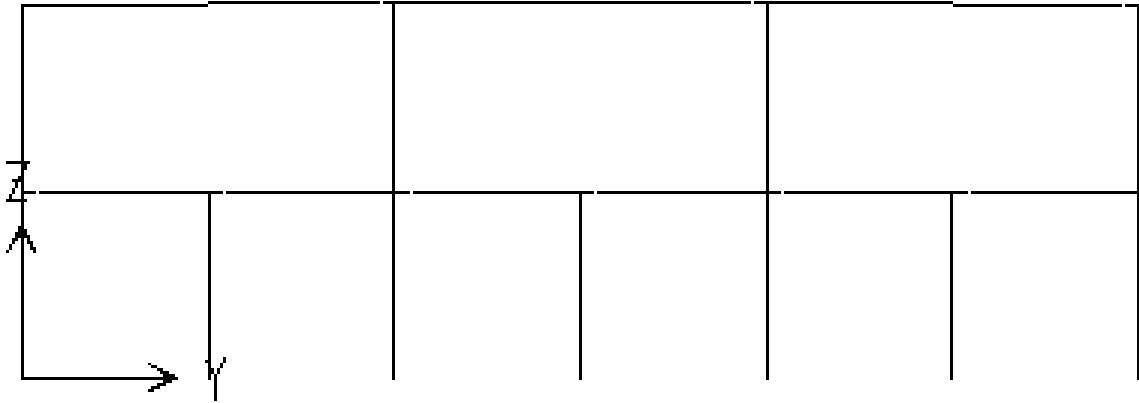


Figure 4.1. SAP2000 frame model of Building 1.

Dimensional properties of the frame and beam and column labels can be seen in Figure 4.2 below.

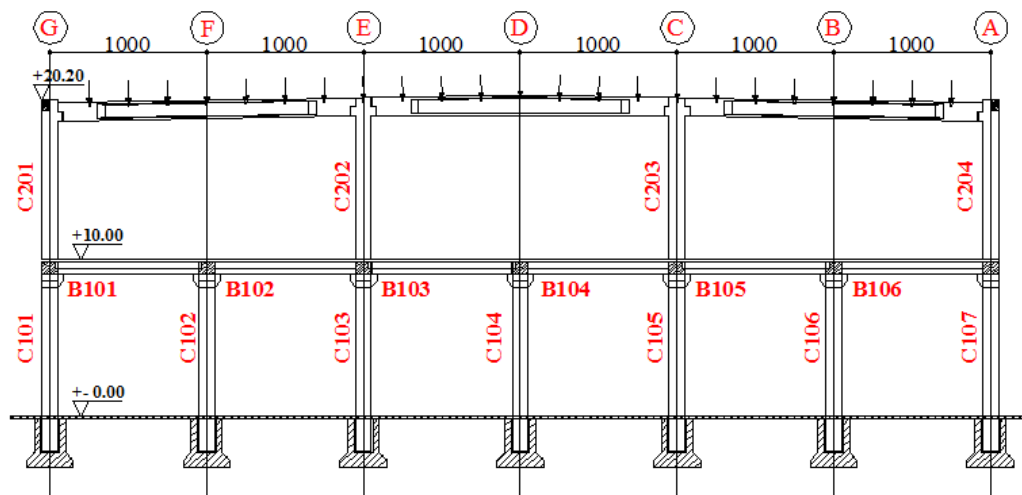


Figure 4.2. Building 1 frame properties, beam and column labels.

Column dimensions are 100/100 where beam dimensions are 80/100 as described in Section 3.

Gravitational loads are defined in the software to perform an initial gravitational load analysis.

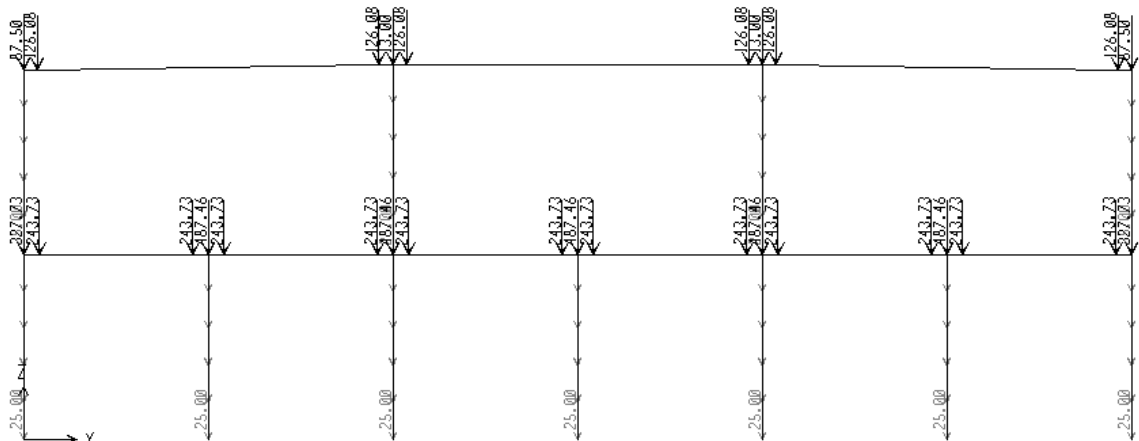


Figure 4.3. SYSTEM1 loads defined.

Note that SYSTEM1 loads are gravitational dead loads that act on the building at Stage 1, as described in Section 1. These loads are due to self weight of columns, pre-cast roof girders, pre-cast purlins, pre-cast storey beams, pre-cast slabs and topping concrete.

Table 4.1. Loads acting on the joints at SIS1 (Stage 1).

Member	Weight Calculation	Weight (kN)
<i>Storey Level (+10.00 Elevation)</i>		
Main Beam	$(0.75\text{m}^2)(25\text{kN/m}^3)(8.96\text{m})/2$	84.00
Topping	$(0.15\text{m})(25\text{kN/m}^3)(5\text{m})(8.96\text{m})/2$	84.00
TT Slabs	$(0.2254\text{m}^2)(25\text{kN/m}^3)(8.96\text{m}/2)(3)$	75.73
Total SIS1 Loading		243.73
<i>Roof Level (+20.00 Elevation)</i>		
Sloped Roof Beam	$(8.50\text{kN/m})(18.96\text{m})/2$	80.58
Purlins	$(0.052\text{m}^2)(25\text{kN/m}^3)(10\text{m})(3.5)$	45.50
Total SIS1 Loading		126.08

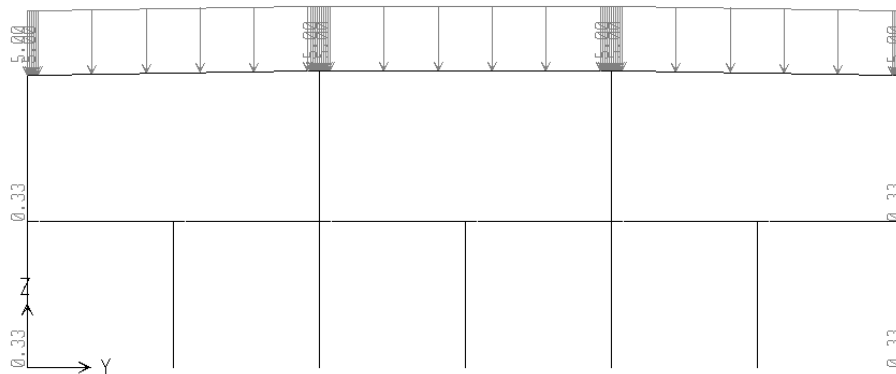


Figure 4.4. G loads defined.

Table 4.3. Total weight calculation of Building 1.

Member	Weight Calculation	Weight (kN)
<i>Storey Level (+10.00 Elevation)</i>		
Columns	$(1.00\text{m}^2)(25\text{kN/m}^3)(5.00\text{m})(3)$	375
	$(1.00\text{m}^2)(25\text{kN/m}^3)(10.00\text{m})(4)$	1,000
TT Slabs	$(0.2254\text{m}^2)(25\text{kN/m}^3)(8.96\text{m}/2)(72)$	1,817.6
Beams	$(0.75\text{m}^2)(25\text{kN/m}^3)(8.96\text{m})(13)$	2,184
Cladding Panel	$(0.10\text{kN/m}^2)(3.33\text{m})(10\text{m})(2)$	7
Topping	$(0.15\text{m})(25\text{kN/m}^3)(10\text{m})(60\text{m})$	2,250
		G = 7,633.7
Live Loading	$(0.80)(25.00\text{kN/m}^2)(10\text{m})(60\text{m})$	12,000
		Q = 12,000
<i>Roof Level (+20.00 Elevation)</i>		
Columns	$(1.00\text{m}^2)(25\text{kN/m}^3)(5\text{m})(4)$	500
Roof Beams	$(8.50\text{kN/m})(18.96\text{m})(3)$	483.5
Purlins	$(0.052\text{m}^2)(25\text{kN/m}^3)(10\text{m})(23)$	299
Perimeter Beams	$(0.35\text{m}^2)(25\text{kN/m}^3)(10\text{m})(2)$	175
Cladding Panel	$(0.10\text{kN/m}^2)(3.33\text{m})(5\text{m})(2)$	3.5
Coating	$(0.50\text{kN/m}^2)(10\text{m})(60\text{m})$	300
		G = 1,761
Snow	$(0.75\text{kN/m}^2)(10\text{m})(60\text{m})$	450
		Q = 450

$$W1 = G + nQ = 7,633\text{kN} + (0.8)(12,000\text{kN}) = 17,233.6\text{kN}$$

$$W2 = G + nQ = 1,761\text{kN} + (0.30)(450\text{kN}) = 1,896\text{kN}$$

- Equivalent Seismic Load Analysis

$$W = 19,129.6\text{kN}$$

$$A(T_1) = A_0 IS(T)$$

$$A_0 = 0.40$$

$$I = 1.0$$

$$S(T) = (2.5)(0.60/1.236)^{0.8} = 1.402$$

Equation 2.16 yields:

$$V_t = 10,730.3\text{kN}$$

$$V_t = \Delta F_N + \Sigma F_i$$

$$\Delta F_N = 0.0075NV_t = (0.0075)(2)(10,730.3\text{kN}) = 161\text{kN}$$

$$F_i = \frac{(V_t - \Delta F_N)(w_i)(h_i)}{\sum w_j * h_j}$$

$$F_1 = (10,730.3kN - 161kN)(0.82) = 8,666.8kN$$

$$F_2 = (10,730.3kN - 161kN)(0.18) + 161kN = 2,063.5kN$$

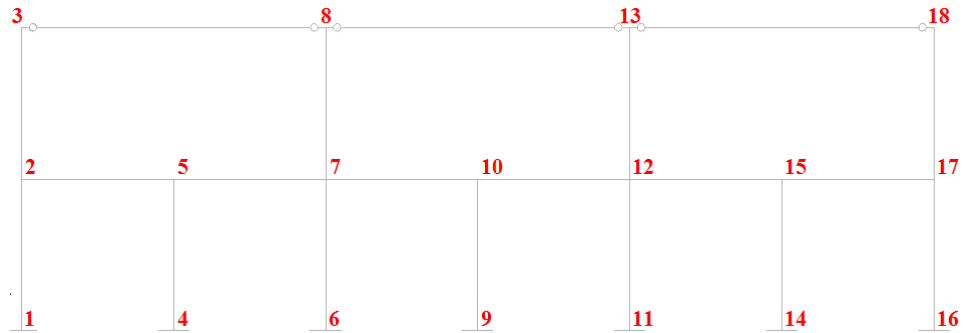


Figure 4.6. Joint labels for Building 1.

Table 4.4. Seismic forces acting on joints of Building 1.

Joint	Mass(kN*s ² /m)	Seismic Force(kN)
First-Storey		
2	167.61	826.9
5	279.19	1,377.4
7	291.94	1,440.3
10	279.19	1,377.4
12	291.94	1,440.3
15	279.19	1,377.4
17	167.61	826.9
Total	1,756.7	8,666.6
Second-Storey		
3	42.08	449.3
8	54.56	582.5
13	54.56	582.5
18	42.08	449.3
Total	193.28	2,063.6

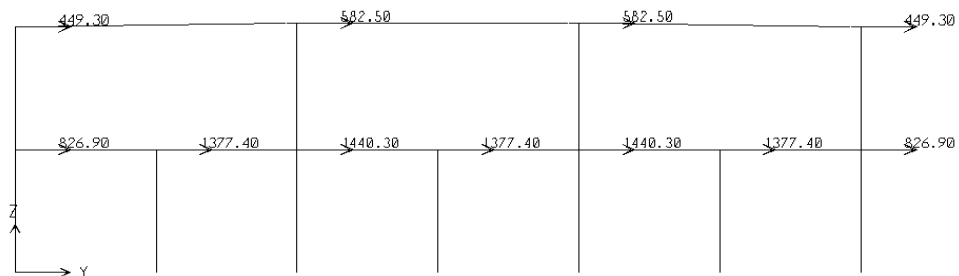


Figure 4.7. Seismic forces defined in SAP2000 for Building 1.

4.1.3 Analysis Results for Building 1

Analysis results due to seismic loads for Building 1 are presented in the figures.

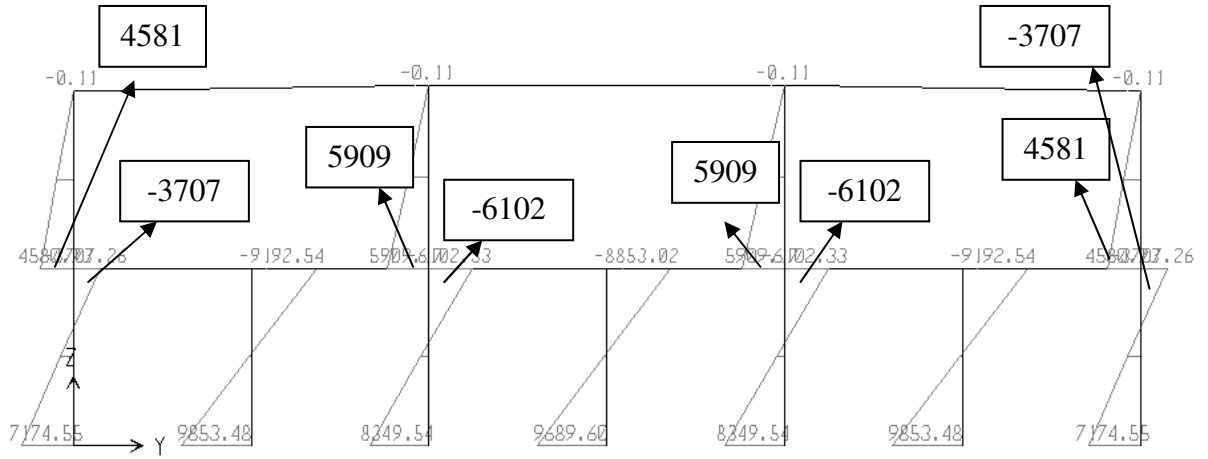


Figure 4.8. Moment diagram for Building 1 / M2-2 – E.

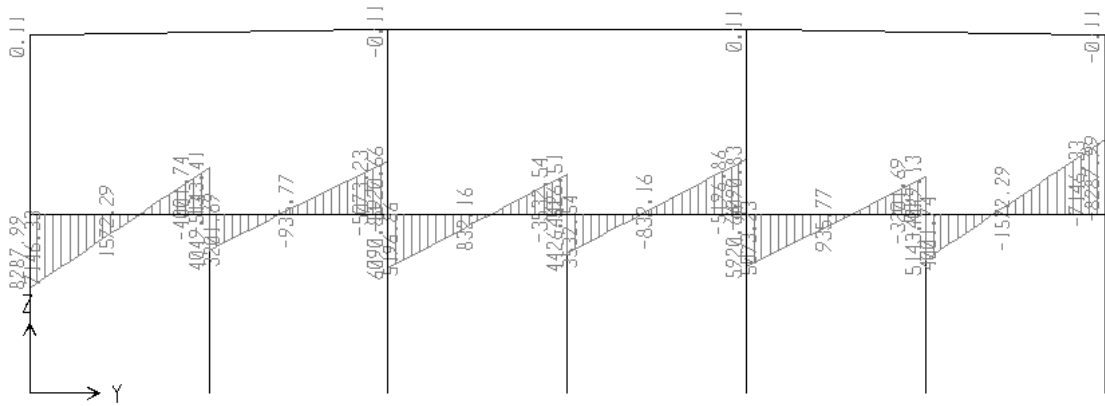


Figure 4.9. Moment diagram for Building 1 / M3-3 – E.

4.1.4 Performance Analysis of Building 1 Beams

Linear elastic performance analysis of beam members of Building 1 is done using the methods described in Section 2.6.(iv) of this study. As an example, performance analysis of left end of B101 member is presented here. Performance analysis results for all beam members are presented in Table 4.6.

It should be noted that since all the beams are designed according to TSC2007, “confinement” effect, which is a parameter in TSC2007 Table 7.2, is present for all members.

- Evaluation of Ductility for B101

$$V_e = V_{dy} \pm (M_{pi} + M_{pj}) / l_n$$

$$V_{dy} = 602.1kN$$

$$M_{ri}^+ = M_{rj}^+ = 3496kNm$$

$$M_{ri}^- = M_{rj}^- = 3121kNm$$

$$V_e = 602.1kN + ((3496kNm + 3121kNm) / 8.96m) = 1,340.6kN$$

$$V_e < V_{e,d} = 1,343.1kN \quad (\text{from SAP2000 model})$$

$$V_r = V_{cw} + V_c \quad (\text{TS-500 Eq. 8.3})$$

$$V_c = (0.8)(0.65)(0.22)(80)(95) = 869.4kN$$

$$V_{cw} = 4\phi 12/10 = 4 * 1.13 * 42 * 95/10 = 1804kN$$

$$V_r = 869.4kN + 1804kN = 2,673.4kN$$

$V_r > V_e$ therefore B101 is ductile.

Determination of ρ_s :

$$\rho = 87.7cm^2 / (80cm * 93cm) = 0.012$$

$$A_s = (3)(8.01cm^2) + (5)(5.31cm^2) + (10)(0.987cm^2)(0.85 * 1860MPa / 420MPa) = 87.7cm^2$$

$$\rho' = 80.4cm^2 / (80cm * 95cm) = 0.0106$$

$$\rho_b = 0.0209$$

$$(\rho - \rho') / \rho_b = 0.067 \sim 0.00$$

$$V_e / (b_w d f_{ctm}) = 1,343.1kN / (80cm)(95cm)(0.22kN/cm^2) = 0.803 > 0.65, < 1.30$$

By linear interpolation:

MN : 2.882

GV : 6.529

GÇ : 9.529

- Determination of r :

$$M_e = 7604 \text{ kNm}$$

$$M_r = 3496 \text{ kNm}$$

$$M_{G+Q} = 0 \text{ kNm}$$

$$r = 7604 \text{ kNm} / (3496 \text{ kNm} - 0 \text{ kNm}) = 2.175$$

Table 4.5. Ductility of Building 1 beams.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
B101	1341	1343	1341	2673	Ductile
B102	1341	997	997	2673	Ductile
B103	1341	997	997	2673	Ductile
B104	1341	997	997	2673	Ductile
B105	1341	997	997	2673	Ductile
B106	1341	1343	1341	2673	Ductile

Table 4.6. Performance evaluation of Building 1 beams.

Member	M_G	M_E	M_R	$V_E/b_w d$ f_{ctm}	$(\rho - \rho')/\rho_b$	r	r_s			D.L.
							MN	GV	GC	
B101-L	0	7604	3496	0.803	0.067	2.18	2.88	6.53	9.53	MDL
B101-R	-634	-4460	3121	0.596	0.067	1.79	3.00	7.00	10.0	MDL
B102-L	0	3549	3496	0.596	0.067	1.02	3.00	7.00	10.0	MDL
B102-R	-613	-5413	3121	0.596	0.067	2.16	3.00	7.00	10.0	MDL
B103-L	0	5561	3496	0.596	0.067	1.59	3.00	7.00	10.0	MDL
B103-R	-609	-3894	3121	0.596	0.067	1.55	3.00	7.00	10.0	MDL
B104-L	0	3899	3496	0.596	0.067	1.12	3.00	7.00	10.0	MDL
B104-R	-638	-5564	3121	0.596	0.067	2.24	3.00	7.00	10.0	MDL
B105-L	0	5416	3496	0.596	0.067	1.55	3.00	7.00	10.0	MDL
B105-R	-623	-3549	3121	0.596	0.067	1.42	3.00	7.00	10.0	MDL
B106-L	0	4463	3496	0.596	0.067	1.28	3.00	7.00	10.0	MDL
B106-R	-356	-7611	3121	0.803	0.067	2.75	2.88	6.53	9.53	MDL

4.1.5 Performance Analysis of Building 1 Columns

Linear elastic performance analysis of column members of Building 1 is done using the methods described in Section 2.6.(iv) of this study. As an example, performance analysis of top end of C101 member shall be presented here. Performance analysis results for all column members are presented in Table 4.10.

It should be noted that since all the columns are designed according to TSC2007, “confinement” effect, which is a parameter in TSC2007 Table 7.3, is present for all members.

Moment-Interaction (P-M) diagrams are obtained using XTRACT [13] sectional analysis software by using appropriate material models specified in TSC2007.

- Determination of r for C101 upper end

Below can be found Moment-Interaction Diagram for C101 upper end, analysed utilizing XTRACT.

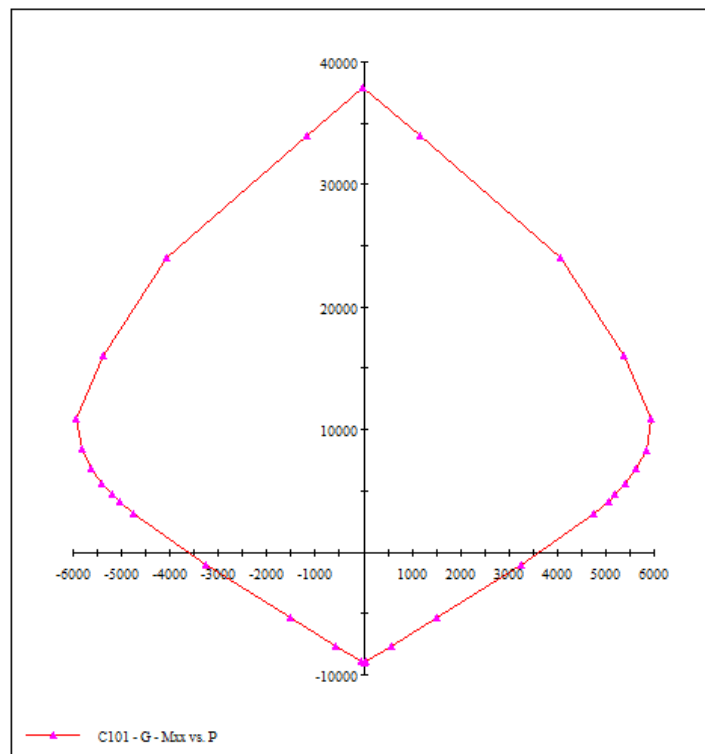


Figure 4.10. Moment-interaction diagram of Building 1 C101 upper section.

$$N_D = 1881kN, M_D = 556kNm$$

$$N_E = -1343kN, M_E = -3707kNm / \Delta N_E = 3,224kN, \Delta M_E = 4,263kNm$$

Equation for nominal forces :

$$y = 1881 + 0.7563*(x-556)$$

Equation for PM boundary :

$$y = 4476 - 2.878*(4677+x)$$

At the intersecting point,

$$1881 + 0.7563x - 420.5 = 4476 - 13460 - 2.878x$$

$$x = -2873.9, y = -713$$

Therefore,

$$N_K = -713kN, M_K = -2873.9kNm$$

$$r = M_E/M_A = 4263/(2873.9+556) = 1.243$$

$$= N_E/N_A = 3224/(1881+713) = 1.243$$

- Determination of r_s for C101 upper end

Determination of ductility:

Check if TSC 2007 Eq. 3.3 is satisfied:

$$M_{ri} = 3496kNm (+EY), M_{ri} = 3121kNm (-EY)$$

$$N_{ra,C102} = 465kN \rightarrow M_{ra,C102} = 3654kNm$$

$$N_{ri,C101} = -363kN \rightarrow M_{ri,C101} = 3334kNm$$

$$3654kNm + 3334kNm = 6988kNm > 1.2 * 3496kNm = 4195kNm$$

Therefore, TSC 2007 Eq. 3.3 is satisfied.

$$M_{ri} = 3496kNm$$

$$M_{Ea,C102} = 4581kNm$$

$$M_{Eü,C101} = -3707kNm$$

$$M_{a,C102} = 3496kNm * 4581/8288 = 1932kNm$$

$$M_{ü,C101} = 3496kNm * 3707/8288 = 1564kNm$$

$$M_{a,C101} = 2936kNm (N=809kN)$$

$$V_E = (1564kNm + 2936kNm)/9m = 500kN < V_{EY} = 1088kN$$

$$V_E = 500kN$$

$$V_r = V_c + V_w \quad \text{(TS-500, Eq. 8.3)}$$

$$V_c = (0.8)(0.65)(1.9)(1000)(950)(1 - 0.3 * 363000/(1000^2)) = 836kN$$

$$V_w = 6\phi 10/12.5 = 6 * 0.78 * 42 * 95/12.5 = 1494kN$$

$$V_r = 836kN + 1494kN = 2330kN > 554kN$$

Therefore, C101 is ductile.

$$N_K/(A_c f_{cm}) = 713/(100^2 * 3) = 0.024$$

$$V_E/(b_w d f_{ctm}) = 500/(100 \cdot 95 \cdot 19) = 0.277$$

From TSC 2007 Table 7.3,

rs for MN = 3

for GV = 6

for G_C = 8

Table 4.7. Ductility of Building 1 columns.

Member	V _{e1} (kN)	V _{e2} (kN)	V _e (kN)	V _r (kN)	Ductility
C101	500	1088	500	3155	Ductile
C102	955	1905	955	3143	Ductile
C103	758	1445	758	3298	Ductile
C104	923	1854	923	3265	Ductile
C105	758	1445	758	3298	Ductile
C106	894	1905	894	3143	Ductile
C107	402	1088	402	3090	Ductile
C201	-	458	458	3193	Ductile
C202	-	574	574	3229	Ductile
C203	-	574	574	3229	Ductile
C204	-	458	458	3193	Ductile

Table 4.8. Demand/capacity ratios, r, for Building 1 columns.

Member	M _D	N _D	M _E	N _E	M _K	N _K	r
C101-B	371	1345	7175	-1343	396	2774	2.83
C101-T	556	1881	-3707	-1343	-2874	-713	1.24
C102-B	171	2038	9854	346	3629	1434	2.80
C102-T	-329	1788	-9193	346	3560	1262	2.74
C103-B	-126	2684	8350	-54	2794	1840	3.25
C103-T	280	2434	-6102	-55	-4459	586	1.35
C104-B	164	2029	9690	0	3576	1302	2.79
C104-T	-315	1779	-8853	0	-5554	2875	1.75
C105-B	125	2684	8350	54	2791	1831	3.09
C105-T	-280	2434	-6102	-54	-3762	946	1.67
C106-B	159	2047	9854	-346	3540	1213	2.87
C106-T	-303	1797	-9193	-346	-3469	1034	2.81
C107-B	-177	2153	7175	1343	3312	1776	2.15
C107-T	-651	1899	-3707	1343	-3595	1363	1.04
C201-B	98	538	4581	0	3524	127	1.31
C202-B	170	664	5909	0	4975	108	1.19
C203-B	139	664	5909	0	4975	108	1.19
C204-B	99	538	4581	0	3524	127	1.31

Table 4.9. Demand/capacity ratio limits, r_s , and Damage Levels for Building 1 columns.

Member	$N_K/A_c f_{cm}$	$V_E/b_w d f_{ctm}$	r_s			r	D.L.
			MN	GV	GC		
C101-B	0.092	0.277	3.00	6.00	8.00	2.83	MDL
C101-T	-0.024	0.277	3.00	6.00	8.00	1.24	MDL
C102-B	0.048	0.529	3.00	6.00	8.00	2.80	MDL
C102-T	0.042	0.529	3.00	6.00	8.00	2.74	MDL
C103-B	0.061	0.420	3.00	6.00	8.00	3.25	VDL
C103-T	0.020	0.420	3.00	6.00	8.00	1.35	MDL
C104-B	0.043	0.511	3.00	6.00	8.00	2.79	MDL
C104-T	0.096	0.511	3.00	6.00	8.00	1.75	MDL
C105-B	0.061	0.420	3.00	6.00	8.00	3.09	VDL
C105-T	0.032	0.420	3.00	6.00	8.00	1.67	MDL
C106-B	0.040	0.495	3.00	6.00	8.00	2.87	MDL
C106-T	0.034	0.495	3.00	6.00	8.00	2.81	MDL
C107-B	0.059	0.223	3.00	6.00	8.00	2.15	MDL
C107-T	0.045	0.223	3.00	6.00	8.00	1.04	MDL
C201-B	0.004	0.254	3.00	6.00	8.00	1.31	MDL
C202-B	0.004	0.318	3.00	6.00	8.00	1.19	MDL
C203-B	0.004	0.318	3.00	6.00	8.00	1.19	MDL
C204-B	0.004	0.254	3.00	6.00	8.00	1.31	MDL

Table 4.10. Damage levels of Building 1 columns due to relative storey drifts.

Column	δ_{ji}	h_{ji}	δ_{ji} / h_{ji}	$(\delta_{ji} / h_{ji})_s$			Damage Level
				MN	GV	GC	
C101	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C102	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C103	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C104	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C105	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C106	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C107	16.9	1000	0.0169	0.01	0.03	0.04	VDL
C201	30.6	1000	0.0306	0.01	0.03	0.04	SDR
C202	30.6	1000	0.0306	0.01	0.03	0.04	SDR
C203	30.6	1000	0.0306	0.01	0.03	0.04	SDR
C204	30.6	1000	0.0306	0.01	0.03	0.04	SDR

4.1.6 Evaluation of Ductility of Beam-Column Connections for Building 1

Ductility of beam-column connections are checked according to TSC 2007 Sec. 7.5.2.6 and tabulated in Table 4.11 below.

Table 4.11. Ductility of beam-column connections of Building 1.

Connection	Type	V_e (kN)	V_r (kN)	Ductility
B101/C101-C201	Unsieged	4446	13500	Ductile
B101-B102/C102	Sieged	8560	18000	Ductile
B102-B103/C103-C202	Sieged	8849	18000	Ductile
B103-B104/C104	Sieged	8560	18000	Ductile
B104-B105/C105-C203	Sieged	8849	18000	Ductile
B105-B106/C106	Sieged	8560	18000	Ductile
B106/C107-C204	Unsieged	4446	13500	Ductile

4.2 Performance Evaluation of Building 1A with Linear Elastic Method

Performance evaluation of Building 1A with Linear Elastic Methods is presented in this section. A 2D frame model of the building is prepared in SAP2000 software.

4.2.1 Mathematical Model for Building 1A

Column dimensions for Building 1A are 160/160 where beam dimensions are 80/100 as described in Section 3.

Gravitational loads defined in the software are identical to Building 1 except for self weights of columns and shall not be presented here.

4.2.2 Seismic Analysis for Building 1A

An initial gravitational load analysis is performed and section rigidities to be used in seismic analysis are determined.

Table 4.12. Section rigidities to be used in Building 1A.

Column	N_D (kN)	$N_D/(A_c * f_{cm})$	$(EI)_e/(EI)_o$
C101	2135	0.028	0.40
C201	927	0.012	0.40
C102	3268	0.043	0.40
C103	4300	0.056	0.40
C202	1067	0.014	0.40
C104	3228	0.042	0.40
C105	4303	0.056	0.40
C203	1066	0.014	0.40
C106	3189	0.042	0.40
C107	3748	0.049	0.40
C204	928	0.012	0.40

Table 4.13. Total weight calculation of Building 1A.

Member	Weight Calculation	Weight (kN)
<i>Storey Level (+7.00 Elevation)</i>		
Columns	$(2.56m^2)(25kN/m^3)(5.00m)(3)$	960
	$(2.56m^2)(25kN/m^3)(10.00m)(4)$	2,560
TT Slabs	$(0.2254m^2)(25kN/m^3)(8.96m/2)(72)$	1,817.6
Beams	$(0.75m^2)(25kN/m^3)(8.96m)(13)$	2,184
Cladding Panel	$(0.10kN/m^2)(3.33m)(10m)(2)$	7
Topping	$(0.15m)(25kN/m^3)(10m)(60m)$	2,250
		G = 9,778.6
Live Loading	$(0.80)(25.00kN/m^2)(10m)(60m)$	12,000
		Q = 12,000
<i>Roof Level (+14.00 Elevation)</i>		
Columns	$(2.56m^2)(25kN/m^3)(5m)(4)$	1,280
Roof Beams	$(8.50kN/m)(18.96m)(3)$	483.5
Purlins	$(0.052m^2)(25kN/m^3)(10m)(23)$	299
Perimeter Beams	$(0.35m^2)(25kN/m^3)(10m)(2)$	175
Cladding Panel	$(0.10kN/m^2)(3.33m)(5m)(2)$	3.5
Coating	$(0.50kN/m^2)(10m)(60m)$	300
		G = 2,541
Snow	$(0.75kN/m^2)(10m)(60m)$	450
		Q = 450

$$W1 = G + nQ = 9,778.6kN + (0.8)(12,000kN) = 19,378.6kN$$

$$W2 = G + nQ = 2,541kN + (0.30)(450kN) = 2,676kN$$

- Equivalent Seismic Load Analysis

$$W = 22,054.6kN$$

$$A(T_1) = A_0 IS(T)$$

$$A_0 = 0.40$$

$$I = 1.0$$

$$S(T) = (2.5)(0.60/0.762)^{0.8} = 2.065$$

Equation 2.16 yields:

$$V_t = 18,216.1kN$$

$$V_t = \Delta F_N + \Sigma F_i$$

$$\Delta F_N = 0.0075NV_t = (0.0075)(2)(18,216.1kN) = 273kN$$

$$F_i = \frac{(V_t - \Delta F_N)(w_i)(h_i)}{\Sigma w_j * h_j}$$

$$F_1 = (18,216.1kN - 273kN)(0.784) = 14,059.6kN$$

$$F_2 = (18,216.1kN - 273kN)(0.18) + 273kN = 4,148.7kN$$

Table 4.14. Seismic Forces Acting on Joints of Building 1A.

Joint	Mass(kN*s ² /m)	Seismic Force(kN)
First-Storey		
2	207.37	1,476
5	299.07	2,128.7
7	331.69	2,360.9
10	299.07	2,128.7
12	331.69	2,360.9
15	299.07	2,128.7
17	207.37	1,476
Total	1,975.3	14,059.6
Second-Storey		
3	61.96	942.3
8	74.43	1,132
13	74.43	1,132
18	61.96	942.3
Total	272.78	4,148.7

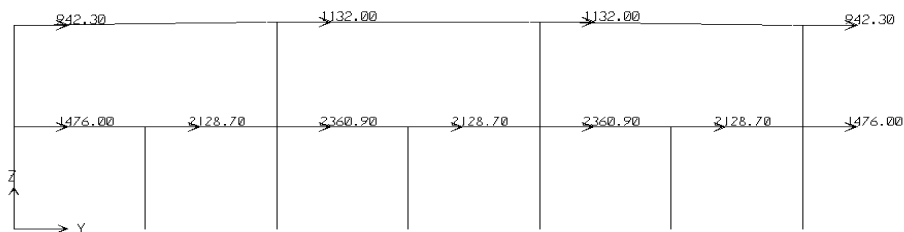


Figure 4.11. Seismic forces defined in SAP2000 for Building 1A.

4.2.3 Analysis Results for Building 1A

Analysis results due to seismic loads for Building 1A are presented in the figures.

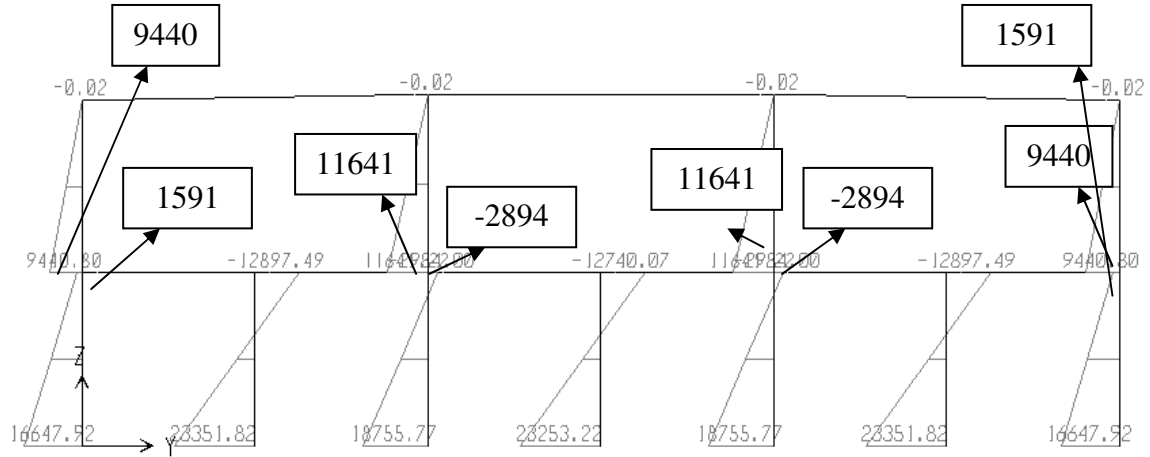


Figure 4.12. Moment diagram for Building 1A / M2-2 – E.

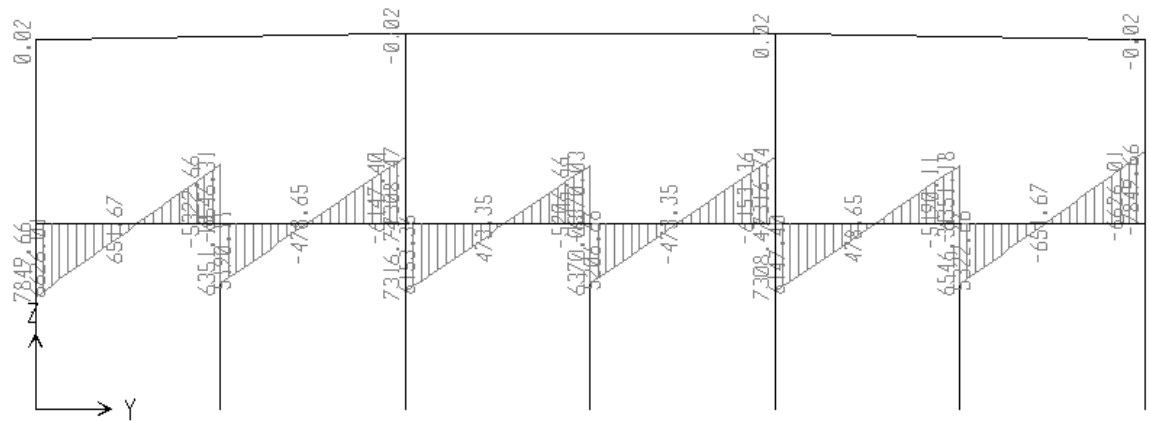


Figure 4.13. Moment diagram for Building 1A / M3-3 – E.

4.2.4 Performance Analysis of Building 1A Beams

Linear elastic performance analysis of beam members of Building 1A is done using the methods described in Section 2.7.(iv) of this study.

An example of performance analysis of beams is given in Section 4.1.4, so only the results shall be presented here.

It should be noted that since all the beams are designed according to TSC2007, “confinement” effect, which is a parameter in TSC2007 Table 7.2, is present for all members.

Table 4.15. Ductility of Building 1A beams.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
B101	1082	1440	1082	2673	Ductile
B102	1082	1366	1082	2673	Ductile
B103	1082	1366	1082	2673	Ductile
B104	1082	1366	1082	2673	Ductile
B105	1082	1366	1082	2673	Ductile
B106	1082	1440	1082	2673	Ductile

Table 4.16. Performance evaluation of Building 1A beams.

Member	M_G	M_E	M_R	V_E/b_w df_{ctm}	$(\rho - \rho')/\rho_b$	r	r_s			D.L.
							MN	GV	GC	
B101-L	0	6696	2325	0.647	0.050	2.88	3.00	7.00	10.0	MDL
B101-R	-404	-5385	2023	0.647	0.050	3.33	3.00	7.00	10.0	VDL
B102-L	0	5246	2325	0.647	0.050	2.26	3.00	7.00	10.0	MDL
B102-R	-400	-6217	2023	0.647	0.050	3.83	3.00	7.00	10.0	VDL
B103-L	0	6216	2325	0.647	0.050	2.67	3.00	7.00	10.0	MDL
B103-R	-389	-5265	2023	0.647	0.050	3.22	3.00	7.00	10.0	VDL
B104-L	0	5270	2325	0.647	0.050	2.27	3.00	7.00	10.0	MDL
B104-R	-396	-6210	2023	0.647	0.050	3.82	3.00	7.00	10.0	VDL
B105-L	0	6216	2325	0.647	0.050	2.67	3.00	7.00	10.0	MDL
B105-R	-393	-5258	2023	0.647	0.050	3.23	3.00	7.00	10.0	VDL
B106-L	0	5395	2325	0.647	0.050	2.32	3.00	7.00	10.0	MDL
B106-R	-340	-6698	2023	0.647	0.050	3.98	3.00	7.00	10.0	VDL

4.2.5 Performance Analysis of Building 1A Columns

Linear elastic performance analysis of column members of Building 1A is done using the methods described in Section 2.7.(iv) of this study.

Performance analysis results for all column members are presented in Table 4.15. It should be noted that since all the columns are designed according to TSC2007, “confinement” effect, which is a parameter in TSC2007 Table 7.3, is present for all members.

Moment-Interaction (P-M) diagrams are obtained using XTRACT [13] sectional analysis software by using appropriate material models specified in TSC2007.

Table 4.17. Ductility of Building 1A columns.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
C101	1008	1506	1008	5989	Ductile
C102	2121	3625	2121	6160	Ductile
C103	1304	2174	1304	6466	Ductile
C104	1995	3599	1995	6285	Ductile
C105	1418	2174	1418	6467	Ductile
C106	2100	3624	2100	4901	Ductile
C107	1216	1506	1216	4995	Ductile
C201	-	944	944	5981	Ductile
C202	-	1130	1130	6164	Ductile
C203	-	1130	1130	6164	Ductile
C204	-	944	944	5981	Ductile

Table 4.18. Demand/capacity ratios, r , for Building 1A columns.

Member	M_D	N_D	M_E	N_E	M_K	N_K	r
C101-B	382	2137	16648	-1440	7729	521	2.21
C101-T	833	2298	1591	-1440	-	-	1.00
C102-B	279	2417	23352	74	8444	1588	2.83
C102-T	-569	1777	-12898	74	-7893	765	1.68
C103-B	230	3480	18756	0	8651	1898	2.20
C103-T	514	2839	-2984	0	-	-	1.00
C104-B	270	2416	23253	0	8424	1559	2.82
C104-T	-560	1776	-12740	0	-7857	712	1.67
C105-B	230	3479	18756	0	8651	1898	2.20
C105-T	-514	2839	-2984	0	-	-	1.00
C106-B	268	2420	23351	-74	8412	1540	2.83
C106-T	-554	1781	-12897	-74	-7839	685	1.69
C107-B	-387	2137	16648	1440	8566	1771	1.90
C107-T	-833	2298	1591	1440	-	-	1.00
C201-B	-175	928	9440	0	7505	187	1.25
C202-B	261	1066	11641	0	7631	375	1.54
C203-B	244	1066	11641	0	7631	375	1.54
C204-B	175	928	9440	0	7509	193	1.26

Table 4.19. Demand/capacity ratio limits, r_s , and damage levels for Building 1A columns.

Member	$N_K/A_c f_{cm}$	$V_E/b_w d f_{ctm}$	r_s			r	D.L.
			MN	GV	GC		
C101-B	0.007	0.214	3.00	6.00	8.00	2.21	MDL
C101-T	0.000	0.214	3.00	6.00	8.00	1.00	MDL
C102-B	0.020	0.450	3.00	6.00	8.00	2.83	MDL
C102-T	0.010	0.450	3.00	6.00	8.00	1.68	MDL
C103-B	0.025	0.277	3.00	6.00	8.00	2.20	MDL
C103-T	0.000	0.277	3.00	6.00	8.00	1.00	MDL
C104-B	0.020	0.423	3.00	6.00	8.00	2.82	MDL
C104-T	0.009	0.423	3.00	6.00	8.00	1.67	MDL
C105-B	0.025	0.301	3.00	6.00	8.00	2.20	MDL
C105-T	0.000	0.301	3.00	6.00	8.00	1.00	MDL
C106-B	0.020	0.446	3.00	6.00	8.00	2.83	MDL
C106-T	0.009	0.446	3.00	6.00	8.00	1.69	MDL
C107-B	0.023	0.258	3.00	6.00	8.00	1.90	MDL
C107-T	0.000	0.258	3.00	6.00	8.00	1.00	MDL
C201-B	0.002	0.200	3.00	6.00	8.00	1.25	MDL
C202-B	0.005	0.240	3.00	6.00	8.00	1.54	MDL
C203-B	0.005	0.240	3.00	6.00	8.00	1.54	MDL
C204-B	0.003	0.200	3.00	6.00	8.00	1.26	MDL

Table 4.20. Damage levels of Building 1A columns due to relative storey drifts.

Column	δ_{ji}	h_{ji}	δ_{ji} / h_{ji}	$(\delta_{ji} / h_{ji})_s$			Damage Level
				MN	GV	GC	
C101	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C102	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C103	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C104	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C105	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C106	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C107	8.42	1000	0.0084	0.01	0.03	0.04	MDL
C201	17.66	1000	0.0177	0.01	0.03	0.04	VDR
C202	17.66	1000	0.0177	0.01	0.03	0.04	VDR
C203	17.66	1000	0.0177	0.01	0.03	0.04	VDR
C204	17.66	1000	0.0177	0.01	0.03	0.04	VDR

4.2.6 Evaluation of Ductility of Beam-Column Connections for Building 1A

Ductility of beam-column connections are checked according to TSC 2007 Sec. 7.5.2.6 and tabulated in Table 4.21.

Table 4.21. Ductility of beam-column connections of Building 1A.

Connection	Type	V_e (kN)	V_r (kN)	Ductility
B101/C101-C201	Unsieged	2455	34560	Ductile
B101-B102/C102	Sieged	2732	46080	Ductile
B102-B103/C103-C202	Sieged	5227	46080	Ductile
B103-B104/C104	Sieged	6855	46080	Ductile
B104-B105/C105-C203	Sieged	5227	46080	Ductile
B105-B106/C106	Sieged	2732	46080	Ductile
B106/C107-C204	Unsieged	2455	34560	Ductile

4.3 Performance Evaluation of Building 2 with Linear Elastic Method

Performance evaluation of Building 2 with Linear Elastic Methods is presented in this section. A 2D frame model of the building is prepared in SAP2000 software.

4.3.1 Mathematical Model for Building 2

Column dimensions for Building 2 are 85/85 where beam dimensions are 80/100 as described in Section 3. Gravitational loads defined in the software are identical to Building 1 except for self weights of columns and shall not be presented here.

4.3.2 Seismic Analysis for Building 2

An initial gravitational load analysis is performed and section rigidities to be used in seismic analysis are determined.

Table 4.22. Section rigidities to be used in Building 2.

Column	N_D (kN)	$N_D/(A_c * f_{cm})$	$(EI)_e/(EI)_o$
C101	1089	0.050	0.40
C201	414	0.019	0.40
C102	1075	0.050	0.40
C103	2270	0.105	0.40
C202	538	0.025	0.40
C104	1746	0.081	0.40
C105	2270	0.105	0.40
C203	538	0.025	0.40
C106	949	0.044	0.40
C107	960	0.044	0.40
C204	414	0.019	0.40

Table 4.23. Total weight calculation of Building 2.

Member	Weight Calculation	Weight (kN)
<i>Storey Level (+10.00 Elevation)</i>		
Columns	$(0.7225\text{m}^2)(25\text{kN/m}^3)(5.00\text{m})(3)$	189.7
	$(0.7225\text{m}^2)(25\text{kN/m}^3)(10.00\text{m})(4)$	505.8
TT Slabs	$(0.2254\text{m}^2)(25\text{kN/m}^3)(8.96\text{m}/2)(72)$	1,817.6
Beams	$(0.75\text{m}^2)(25\text{kN/m}^3)(8.96\text{m})(13)$	2,184
Cladding Panel	$(0.10\text{kN/m}^2)(3.33\text{m})(10\text{m})(2)$	7
Topping	$(0.15\text{m})(25\text{kN/m}^3)(10\text{m})(60\text{m})$	2,250
		G = 6,952.1
Live Loading	$(0.80)(25.00\text{kN/m}^2)(10\text{m})(60\text{m})$	12,000
		Q = 12,000
<i>Roof Level (+20.00 Elevation)</i>		
Columns	$(0.7225\text{m}^2)(25\text{kN/m}^3)(5\text{m})(4)$	252.9
Roof Beams	$(8.50\text{kN/m})(18.96\text{m})(3)$	483.5
Purlins	$(0.052\text{m}^2)(25\text{kN/m}^3)(10\text{m})(23)$	299
Perimeter Beams	$(0.35\text{m}^2)(25\text{kN/m}^3)(10\text{m})(2)$	175
Cladding Panel	$(0.10\text{kN/m}^2)(3.33\text{m})(5\text{m})(2)$	3.5
Coating	$(0.50\text{kN/m}^2)(10\text{m})(60\text{m})$	300
		G = 1,512.9
Snow	$(0.75\text{kN/m}^2)(10\text{m})(60\text{m})$	450
		Q = 450

$$W1 = G + nQ = 6,952.1\text{kN} + (0.8)(12,000\text{kN}) = 16,552.1\text{kN}$$

$$W2 = G + nQ = 1,512.9\text{kN} + (0.30)(450\text{kN}) = 1,647.9\text{kN}$$

- Equivalent Seismic Load Analysis:

$$W = 18,200\text{kN}$$

$$A(T_1) = A_0 IS(T)$$

$$A_0 = 0.40$$

$$I = 1.0$$

$$S(T) = (2.5)(0.60/0.93)^{0.8} = 1.761$$

Equation 2.16 yields:

$$V_t = 12,812.8\text{kN}$$

$$V_t = \Delta F_N + \Sigma F_i$$

$$\Delta F_N = 0.0075NV_t = (0.0075)(2)(12,812.8\text{kN}) = 192\text{kN}$$

$$F_i = \frac{(V_t - \Delta F_N)(w_i)(h_i)}{\sum w_j * h_j}$$

$$F_1 = (12,812.8kN - 192kN)(0.834) = 10,525.7kN$$

$$F_2 = (12,812.8kN - 192kN)(0.166) + 192kN = 2,287.1kN$$

Table 4.24. Seismic forces acting on Building 2 joints.

Joint	Mass(kN*s ² /m)	Seismic Force(kN)
First-Storey		
2	155.02	967
5	272.90	1,702.3
7	279.34	1,742.5
10	272.90	1,702.3
12	279.34	1,742.5
15	272.90	1,702.3
17	155.02	967
Total	1,687.4	10,525.9
Second-Storey		
3	35.78	486.9
8	48.26	656.7
13	48.26	656.7
18	35.78	486.9
Total	168.08	2,287.2

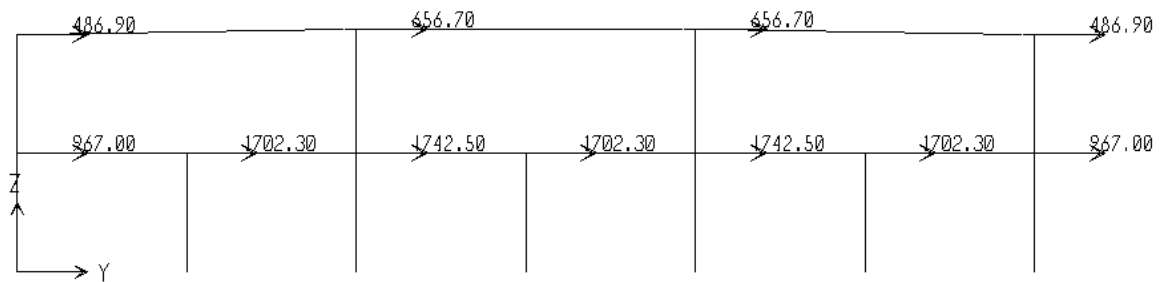


Figure 4.14. Seismic forces defined in SAP2000 for Building 2.

4.3.3 Analysis Results for Building 2

Analysis results due to seismic loads for Building 2 are presented in the following figures.

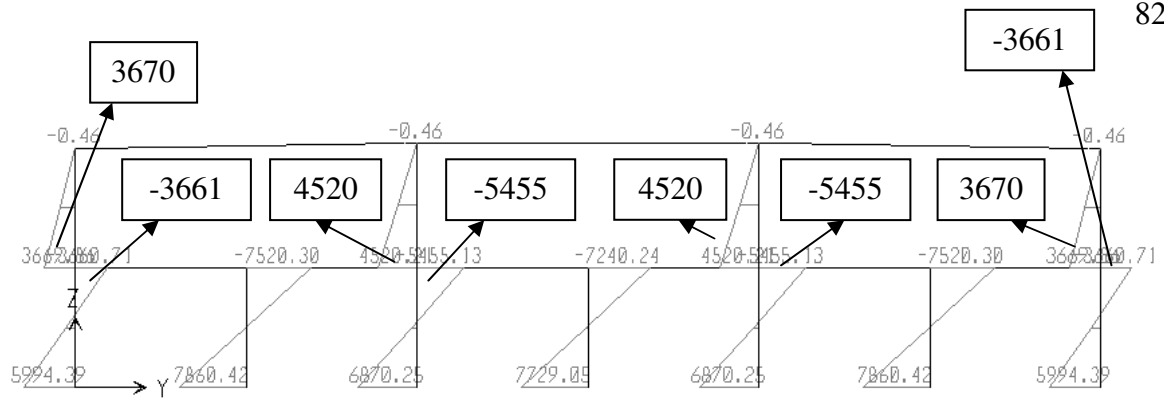


Figure 4.15. Moment diagram for Building 2 / M2-2 – E.

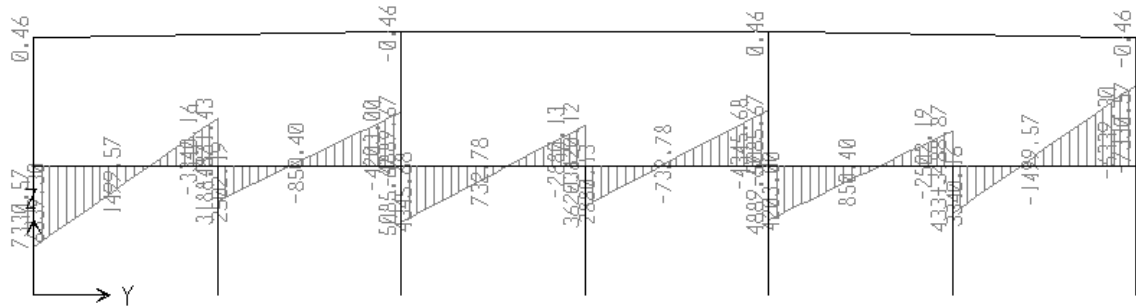


Figure 4.16. Moment diagram for Building 2 / M3-3 – E.

4.3.4 Performance Analysis of Building 2 Beams

Linear elastic performance analysis of beam members of Building 2 is done using the methods described in Section 2.7.(iv) of this study. An example of performance analysis of beams is given in Section 4.1.4, so only the results shall be presented here.

Table 4.25. Ductility of Building 2 beams.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
B101	1282	1166	1166	2673	Ductile
B102	1282	808	808	2673	Ductile
B103	1282	870	870	2673	Ductile
B104	1282	870	870	2673	Ductile
B105	1282	808	808	2673	Ductile
B106	1282	1166	1166	2673	Ductile

Table 4.26. Performance evaluation of Building 2 beams.

Member	M_G	M_E	M_R	V_E/b_w df_{ctm}	$(\rho - \rho')/\rho_b$	r	r_s			D.L.
							MN	GV	GC	
B101-L	0	6816	3088	0.697	0.036	2.21	2.96	6.86	9.86	MDL
B101-R	-700	-3830	3000	0.697	0.036	1.67	2.96	6.86	9.86	MDL
B102-L	0	2870	3088	0.483	0.036	0.92	3.00	7.00	10.0	MDL
B102-R	-682	-4546	3000	0.483	0.036	1.96	3.00	7.00	10.0	MDL
B103-L	0	4711	3088	0.520	0.036	1.53	3.00	7.00	10.0	MDL
B103-R	-677	-3250	3000	0.520	0.036	1.40	3.00	7.00	10.0	MDL
B104-L	0	3250	3088	0.520	0.036	1.05	3.00	7.00	10.0	MDL
B104-R	-700	-4713	3000	0.520	0.036	2.05	3.00	7.00	10.0	MDL
B105-L	0	4551	3088	0.483	0.036	1.47	3.00	7.00	10.0	MDL
B105-R	-691	-2848	3000	0.483	0.036	1.23	3.00	7.00	10.0	MDL
B106-L	0	3830	3088	0.697	0.036	1.24	2.96	6.86	9.86	MDL
B106-R	-352	-6838	3000	0.697	0.036	2.58	2.96	6.86	9.86	MDL

4.3.5 Performance Analysis of Building 2 Columns

Linear elastic performance analysis of column members of Building 2 is done using the methods described in Section 2.7.(iv) of this study. Performance analysis results for all column members are presented in Table 4.25.

It should be noted that since all the columns are designed according to TSC2007, “confinement” effect, which is a parameter in TSC2007 Table 7.3, is present for all members.

Table 4.27. Ductility of Building 2 columns.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
C101	640	1379	640	2433	Ductile
C102	1080	2197	1080	2258	Ductile
C103	961	1761	961	2301	Ductile
C104	1039	2139	1039	2273	Ductile
C105	968	1761	968	2301	Ductile
C106	1018	2197	1018	2219	Ductile
C107	744	1379	744	2297	Ductile
C201	-	524	524	2191	Ductile
C202	-	619	619	2216	Ductile
C203	-	619	619	2216	Ductile
C204	-	524	524	2191	Ductile

Table 4.28. Demand/capacity ratios, r , for Building 2 columns.

Member	M_D	N_D	M_E	N_E	M_K	N_K	r
C101-B	355	1093	5994	-1166	2039	418	3.35
C101-T	605	1771	-3661	-1166	-2129	-111	1.56
C102-B	145	1917	7860	358	2687	1403	3.04
C102-T	-280	1791	-7520	358	2659	1320	3.04
C103-B	-104	2433	6870	-62	1953	1697	3.39
C103-T	235	2307	-5455	-62	-3627	103	1.47
C104-B	138	1905	7729	0	2644	1276	3.03
C104-T	-266	1779	-7240	0	2571	1055	2.74
C105-B	104	2433	6870	62	1978	1776	3.61
C105-T	-235	2307	-5455	62	-3988	693	1.39
C106-B	132	1926	7860	-358	2624	1214	3.21
C106-T	-253	1801	-7520	-358	-2588	1107	3.11
C107-B	-356	1092	5994	1167	2253	1061	2.43
C107-T	-605	1771	-3661	1167	-2609	1375	1.53
C201-B	94	413	3670	0	1966	196	1.91
C202-B	147	538	4520	0	3629	110	1.26
C203-B	114	538	4520	0	3798	88	1.20
C204-B	-94	413	3670	0	2216	160	1.63

Table 4.29. Demand/capacity ratio limits, r_s , and damage levels for Building 2 columns.

Member	$N_K/A_c f_{cm}$	$V_E/b_w d f_{ctm}$	r_s			r	D.L.
			MN	GV	GC		
C101-B	0.019	0.495	3.00	6.00	8.00	3.35	VDL
C101-T	-0.005	0.495	3.00	6.00	8.00	1.56	MDL
C102-B	0.065	0.722	2.95	5.78	7.78	3.04	VDL
C102-T	0.061	0.722	2.95	5.78	7.78	3.04	VDL
C103-B	0.078	0.642	3.00	6.00	8.00	3.39	VDL
C103-T	0.005	0.642	3.00	6.00	8.00	1.47	MDL
C104-B	0.059	0.695	2.97	5.86	7.86	3.03	VDL
C104-T	0.049	0.695	2.97	5.86	7.86	2.74	MDL
C105-B	0.082	0.647	3.00	6.00	8.00	3.61	VDL
C105-T	0.032	0.647	3.00	6.00	8.00	1.39	MDL
C106-B	0.056	0.680	2.98	5.91	7.91	3.21	VDL
C106-T	0.051	0.680	2.98	5.91	7.91	3.11	VDL
C107-B	0.049	0.497	3.00	6.00	8.00	2.43	MDL
C107-T	0.063	0.497	3.00	6.00	8.00	1.53	MDL
C201-B	0.009	0.350	3.00	6.00	8.00	1.91	MDL
C202-B	0.005	0.414	3.00	6.00	8.00	1.26	MDL
C203-B	0.004	0.414	3.00	6.00	8.00	1.20	MDL
C204-B	0.007	0.350	3.00	6.00	8.00	1.63	MDL

Table 4.30. Damage levels of Building 2 columns due to relative storey drifts.

Column	δ_{ji}	h_{ji}	δ_{ji} / h_{ji}	$(\delta_{ji} / h_{ji})_s$			Damage Level
				MN	GV	GÇ	
C101	12.5	700	0.0179	0.01	0.03	0.04	VDL
C102	12.5	700	0.0179	0.01	0.03	0.04	VDL
C103	12.5	700	0.0179	0.01	0.03	0.04	VDL
C104	12.5	700	0.0179	0.01	0.03	0.04	VDL
C105	12.5	700	0.0179	0.01	0.03	0.04	VDL
C106	12.5	700	0.0179	0.01	0.03	0.04	VDL
C107	12.5	700	0.0179	0.01	0.03	0.04	VDL
C201	21.2	700	0.0303	0.01	0.03	0.04	SDL
C202	21.2	700	0.0303	0.01	0.03	0.04	SDL
C203	21.2	700	0.0303	0.01	0.03	0.04	SDL
C204	21.2	700	0.0303	0.01	0.03	0.04	SDL

4.3.6 Evaluation of Ductility of Beam-Column Connections for Building 2

Ductility of beam-column connections are checked according to TSC 2007 Sec. 7.5.2.6 and tabulated in Table 4.26 below.

Table 4.31. Ductility of beam-column connections of Building 2.

Connection	Type	V_e (kN)	V_r (kN)	Ductility
B101/C101-C201	Unsieged	3991	9754	Ductile
B101-B102/C102	Sieged	6704	13005	Ductile
B102-B103/C103-C202	Sieged	8282	13005	Ductile
B103-B104/C104	Sieged	6763	13005	Ductile
B104-B105/C105-C203	Sieged	8282	13005	Ductile
B105-B106/C106	Sieged	6704	13005	Ductile
B106/C107-C204	Unsieged	3991	9754	Ductile

4.4 Performance Evaluation of Building 2A with Linear Elastic Method

Performance evaluation of Building 2A with Linear Elastic Methods is presented in this section. A 2D frame model of the building is prepared in SAP2000 software.

4.4.1 Mathematical Model for Building 2A

Column dimensions for Building 2A are 130/130 where beam dimensions are 80/100 as described in Section 3. Gravitational loads defined in the software are identical to Building 1 except for self weights of columns and shall not be presented here.

4.4.2 Seismic Analysis for Building 2A

An initial gravitational load analysis is performed and section rigidities to be used in seismic analysis are determined.

Table 4.32. Section rigidities to be used in Building 2A.

Column	N_D (kN)	$N_D/(A_c * f_{cm})$	$(EI)_e/(EI)_o$
C101	1139	0.022	0.40
C201	583	0.012	0.40
C102	966	0.019	0.40
C103	2471	0.049	0.40
C202	715	0.014	0.40
C104	1766	0.035	0.40
C105	2471	0.049	0.40
C203	715	0.014	0.40
C106	966	0.019	0.40
C107	1139	0.022	0.40
C204	583	0.011	0.40

Table 4.33. Total weight calculation of Building 2A.

Member	Weight Calculation	Weight (kN)
<i>Storey Level (+10.00 Elevation)</i>		
Columns	$(1.69m^2)(25kN/m^3)(5.00m)(3)$	444
	$(1.69m^2)(25kN/m^3)(10.00m)(4)$	1,183
TT Slabs	$(0.2254m^2)(25kN/m^3)(8.66m/2)(72)$	1,756.8
Beams	$(0.75m^2)(25kN/m^3)(8.66m)(13)$	2,111
Cladding Panel	$(0.10kN/m^2)(3.33m)(10m)(2)$	7
Topping	$(0.15m)(25kN/m^3)(10m)(60m)$	2,250
		G = 7,751.8
Live Loading	$(0.80)(25.00kN/m^2)(10m)(60m)$	12,000
		Q = 12,000
<i>Roof Level (+20.00 Elevation)</i>		
Columns	$(1.69m^2)(25kN/m^3)(5m)(4)$	591.5
Roof Beams	$(8.50kN/m)(18.66m)(3)$	475.8
Purlins	$(0.052m^2)(25kN/m^3)(10m)(23)$	299

Table 4.33cont. Total weight calculation of Building 2A.

Member	Weight Calculation	Weight (kN)
Perimeter Beams	$(0.35\text{m}^2)(25\text{kN/m}^3)(10\text{m})(2)$	175
Cladding Panel	$(0.10\text{kN/m}^2)(3.33\text{m})(5\text{m})(2)$	3.5
Coating	$(0.50\text{kN/m}^2)(10\text{m})(60\text{m})$	300
		G = 1,844.8
Snow	$(0.75\text{kN/m}^2)(10\text{m})(60\text{m})$	450
		Q = 450

$$W1 = G + nQ = 7,751.8\text{kN} + (0.8)(12,000\text{kN}) = 17,351.8\text{kN}$$

$$W2 = G + nQ = 1,844.8\text{kN} + (0.30)(450\text{kN}) = 1,979.8\text{kN}$$

- Equivalent Seismic Load Analysis

$$W = 19,290.6\text{kN}$$

$$A(T_1) = A_0 IS(T)$$

$$A_0 = 0.40$$

$$I = 1.0$$

$$S(T) = 2.5$$

Equation 2.16 yields:

$$V_t = 19,290.6\text{kN}$$

$$V_t = \Delta F_N + \Sigma F_i$$

$$\Delta F_N = 0.0075NV_t = (0.0075)(2)(19,290.6\text{kN}) = 289\text{kN}$$

$$F_i = \frac{(V_t - \Delta F_N)(w_i)(h_i)}{\Sigma w_j * h_j}$$

$$F_1 = (19,290.6\text{kN} - 289\text{kN})(0.814) = 15,467.3\text{kN}$$

$$F_2 = (19,290.6\text{kN} - 289\text{kN})(0.186) + 289\text{kN} = 3,823.3\text{kN}$$

Table 4.34. Seismic forces acting on Building 2A joints.

Joint	Mass(kN*s ² /m)	Seismic Force(kN)
First-Storey		
2	168.86	1,480.1
5	279.35	2,448.6
7	294.42	2,580.7
10	279.35	2,448.6

Table 4.34cont. Seismic forces acting on Building 2A joints.

Joint	Mass(kN*s ² /m)	Seismic Force(kN)
12	294.42	2,580.7
15	279.35	2,448.6
17	168.86	1,480.1
Total	1,764.6	15,467.3
Second-Storey		
3	44.28	838.9
8	56.62	1,072.7
13	56.62	1,072.7
18	44.28	838.9
Total	201.8	3,823.3

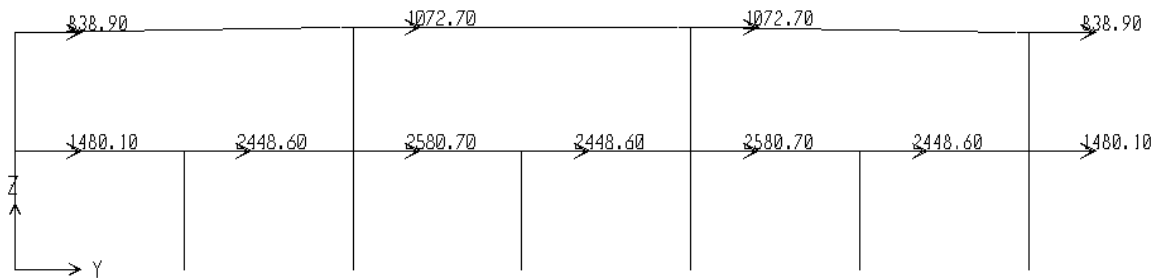


Figure 4.17. Seismic forces defined in SAP2000 for Building 2A.

4.4.3 Analysis Results for Building 2A

Analysis results due to seismic loads for Building 2A are presented in the figures below.

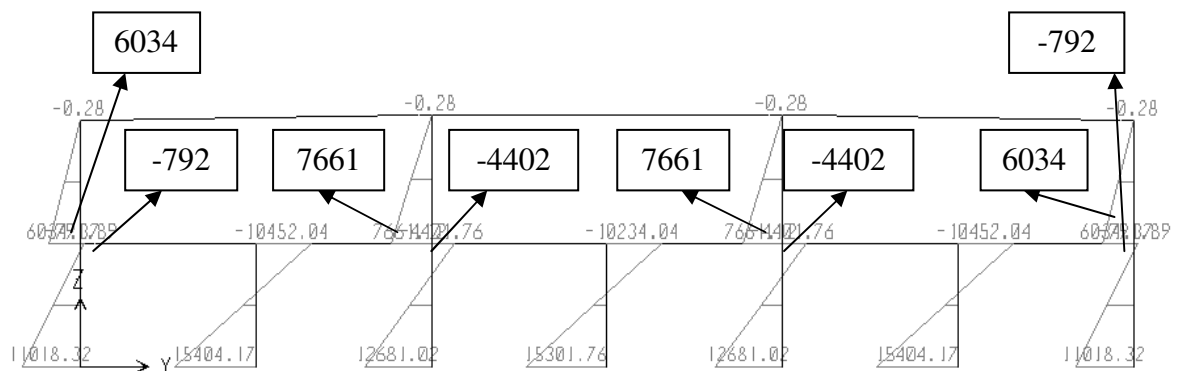


Figure 4.18. Moment diagram for Building 2A / M2-2 – E.

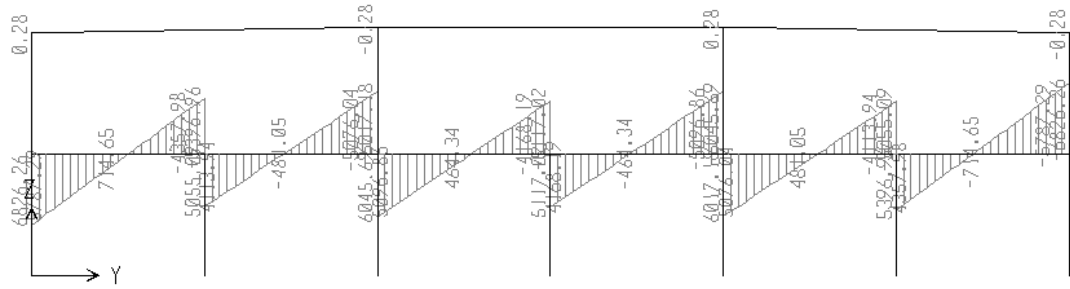


Figure 4.19. Moment diagram for Building 2A / M3-3 – E.

4.4.4 Performance Analysis of Building 2A Beams

Linear elastic performance analysis of beam members of Building 2A is done using the methods described in Section 2.7.(iv) of this study. An example of performance analysis of beams is given in Section 4.1.4, so only the results shall be presented here.

Table 4.35. Ductility of Building 2A beams.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
B101	1084	1222	1084	2673	Ductile
B102	1084	1107	1084	2673	Ductile
B103	1084	1115	1084	2673	Ductile
B104	1084	1115	1084	2673	Ductile
B105	1084	1107	1084	2673	Ductile
B106	1084	1222	1084	2673	Ductile

Table 4.36. Performance evaluation of Building 2A beams.

Member	M_G	M_E	M_R	V_E/b_w df_{ctm}	$(\rho - \rho')/\rho_b$	r	r_s			D.L.
							MN	GV	GC	
B101-L	0	6032	2325	0.648	0.050	2.59	3.00	7.00	10.0	MDL
B101-R	-508	-4602	2023	0.648	0.050	3.04	3.00	7.00	10.0	VDL
B102-L	0	4335	2325	0.648	0.050	1.87	3.00	7.00	10.0	MDL
B102-R	-506	-5298	2023	0.648	0.050	3.49	3.00	7.00	10.0	VDL
B103-L	0	5320	2325	0.648	0.050	2.29	3.00	7.00	10.0	MDL
B103-R	-496	-4391	2023	0.648	0.050	2.88	3.00	7.00	10.0	MDL
B104-L	0	4391	2325	0.648	0.050	1.89	3.00	7.00	10.0	MDL
B104-R	-512	-5320	2023	0.648	0.050	3.52	3.00	7.00	10.0	VDL
B105-L	0	5297	2325	0.648	0.050	2.28	3.00	7.00	10.0	MDL
B105-R	-493	-4335	2023	0.648	0.050	2.83	3.00	7.00	10.0	MDL
B106-L	0	4602	2325	0.648	0.050	1.98	3.00	7.00	10.0	MDL
B106-R	-397	-6032	2023	0.648	0.050	3.71	3.00	7.00	10.0	VDL

4.4.5 Performance Analysis of Building 2A Columns

Linear elastic performance analysis of column members of Building 2A is done using the methods described in Section 2.7.(iv) of this study. Performance analysis results for all column members are presented in Table 4.33 and Table 4.34.

Table 4.37. Ductility of Building 2A columns.

Member	V_{e1} (kN)	V_{e2} (kN)	V_e (kN)	V_r (kN)	Ductility
C101	880	1687	880	3929	Ductile
C102	1709	3694	1709	3929	Ductile
C103	1400	2440	1400	3929	Ductile
C104	1710	3645	1710	3929	Ductile
C105	1400	2440	1400	3929	Ductile
C106	1710	3694	1710	3929	Ductile
C107	892	1687	892	3929	Ductile
C201	-	862	862	3929	Ductile
C202	-	1050	1050	3929	Ductile
C203	-	1050	1050	3929	Ductile
C204	-	862	862	3929	Ductile

Table 4.38. Demand/capacity ratios, r , for Building 2A columns.

Member	M_D	N_D	M_E	N_E	M_K	N_K	r
C101-B	368	1444	11018	-1223	3977	537	2.95
C101-T	802	1951	-792	-1223	-	-	1.00
C102-B	254	2074	15404	115	4493	1526	3.57
C102-T	-523	1779	-10452	115	-4295	1147	2.63
C103-B	201	2784	12681	0	4633	1795	2.82
C103-T	-462	2468	-4402	0	-3885	361	1.15
C104-B	244	2073	15302	0	4474	1490	3.56
C104-T	-510	1777	-10234	0	-4266	1091	2.59
C105-B	202	2784	12681	0	4633	1795	2.82
C105-T	-469	0	-4402	0	-3872	335	1.16
C106-B	239	2080	15404	-115	4463	1469	3.59
C106-T	523	1779	-10452	-115	-4286	1129	2.92
C107-B	-368	1443	11018	1223	4401	1351	2.39
C107-T	-802	1952	-792	1223	-	-	1.00
C201-B	-155	583	6034	0	3807	210	1.56
C202-B	246	715	7661	0	3887	364	2.04
C203-B	219	715	7661	0	3887	364	2.04
C204-B	155	583	6034	0	3807	210	1.56

Table 4.39. Demand/capacity ratio limits, r_s , and damage levels for Building 2A columns.

Member	$N_K/A_c f_{cm}$	$V_E/b_w d f_{ctm}$	r_s			r	D.L.
			MN	GV	GC		
C101-B	0.011	0.285	3.00	6.00	8.00	2.95	MDL
C101-T	0.000	0.285	3.00	6.00	8.00	1.00	MDL
C102-B	0.030	0.554	3.00	6.00	8.00	3.57	VDL
C102-T	0.023	0.554	3.00	6.00	8.00	2.63	MDL
C103-B	0.035	0.453	3.00	6.00	8.00	2.82	MDL
C103-T	0.007	0.453	3.00	6.00	8.00	1.15	MDL
C104-B	0.029	0.554	3.00	6.00	8.00	3.56	VDL
C104-T	0.022	0.554	3.00	6.00	8.00	2.59	MDL
C105-B	0.035	0.453	3.00	6.00	8.00	2.82	MDL
C105-T	0.007	0.453	3.00	6.00	8.00	1.16	MDL
C106-B	0.277	0.554	3.00	6.00	8.00	3.59	VDL
C106-T	0.213	0.554	3.00	6.00	8.00	2.92	MDL
C107-B	0.255	0.289	3.00	6.00	8.00	2.39	MDL
C107-T	-	0.289	3.00	6.00	8.00	1.00	MDL
C201-B	0.004	0.279	3.00	6.00	8.00	1.56	MDL
C202-B	0.007	0.340	3.00	6.00	8.00	2.04	MDL
C203-B	0.007	0.340	3.00	6.00	8.00	2.04	MDL
C204-B	0.004	0.279	3.00	6.00	8.00	1.56	MDL

Table 4.40. Damage levels of Building 2A columns due to relative storey drifts.

Column	δ_{ji}	h_{ji}	δ_{ji} / h_{ji}	$(\delta_{ji} / h_{ji})_s$			Damage Level
				MN	GV	GC	
C101	5.85	700	0.0084	0.01	0.03	0.04	MDL
C102	5.85	700	0.0084	0.01	0.03	0.04	MDL
C103	5.85	700	0.0084	0.01	0.03	0.04	MDL
C104	5.85	700	0.0084	0.01	0.03	0.04	MDL
C105	5.85	700	0.0084	0.01	0.03	0.04	MDL
C106	5.85	700	0.0084	0.01	0.03	0.04	MDL
C107	5.85	700	0.0084	0.01	0.03	0.04	MDL
C201	11.54	700	0.0165	0.01	0.03	0.04	VDL
C202	11.54	700	0.0165	0.01	0.03	0.04	VDL
C203	11.54	700	0.0165	0.01	0.03	0.04	VDL
C204	11.54	700	0.0165	0.01	0.03	0.04	VDL

4.4.6 Evaluation of Ductility of Beam-Column Connections for Building 2A

Ductility of beam-column connections are checked according to TSC 2007 Sec. 7.5.2.6 and tabulated in Table 4.41.

Table 4.41. Ductility of beam-column connections of Building 2A.

Connection	Type	V_e (kN)	V_r (kN)	Ductility
B101/C101-C201	Unsieged	2881	22815	Ductile
B101-B102/C102	Sieged	4104	30420	Ductile
B102-B103/C103-C202	Sieged	7583	30420	Ductile
B103-B104/C104	Sieged	6020	30420	Ductile
B104-B105/C105-C203	Sieged	7583	30420	Ductile
B105-B106/C106	Sieged	4104	30420	Ductile
B106/C107-C204	Unsieged	2881	22815	Ductile

5. PERFORMANCE ANALYSIS OF STRUCTURES WITH NONLINEAR STATIC METHOD (PUSHOVER ANALYSIS)

This chapter of the study describes in detail the methodology of performance analysis of the four structures using nonlinear static method, also known as the “*incremental pushover analysis*”. Since the specifications of TSC2007 for performance analysis with nonlinear analysis methods are described in Section 2.7 of this study, mainly the numerical parameters and results, with relevant references to Section 2.7, shall be presented in this chapter.

In this section, in addition to the four frames analyzed utilizing linear elastic methods, two additional frames, Building 1B and Building 2B are analyzed. These two frames have identical dimensions with Building 1 and 2, respectively. The only difference is the longitudinal reinforcement provided at one-storey columns, namely C102, C104 and C106.

It is crucial to note that although initial analysis and design of the four buildings are performed using 3D mathematical models, accounting for additional eccentricities and effects of seismic forces in orthogonal directions simultaneously, planar analyses are performed in performance evaluation, as was the case for linear elastic methods.

2D mathematical models representing the frames are prepared and pushover analyses are performed by utilizing SAP2000 [12] structural analysis software.

Analysis procedure and details, describing in detail how pushover analysis is performed, are presented in Section 5.1. Nonlinear static performance analyses for Building 1, Building 1A, Building 2, Building 2A are presented in Section 5.2, Section 5.3, Section 5.4 and Section 5.5, respectively. Also, buildings 1B and 2B, representing identical pre-cast structures with strengthened one-storey columns (C102, C104 and C106) as a strategy to improve seismic performance of the buildings in light of the results obtained, are analyzed in Section 5.6 and Section 5.7, respectively.

5.1 Nonlinear Static Performance Analysis Methodology

Nonlinear static performance analysis is performed utilizing SAP2000 structural analysis software. The buildings are modeled in SAP2000 as frames. A representative model is shown in the figure below.

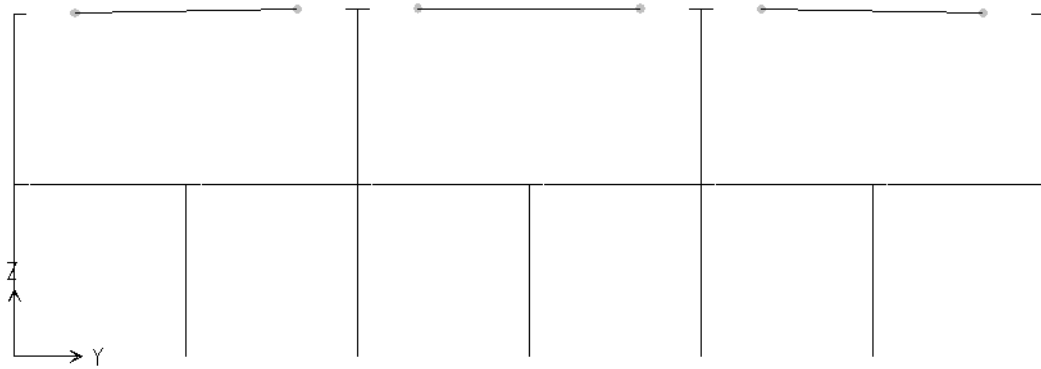


Figure 5.1. 2D mathematical model for pushover analysis.

Plastic hinges are assigned to the sections where plastic behaviour is likely to occur.

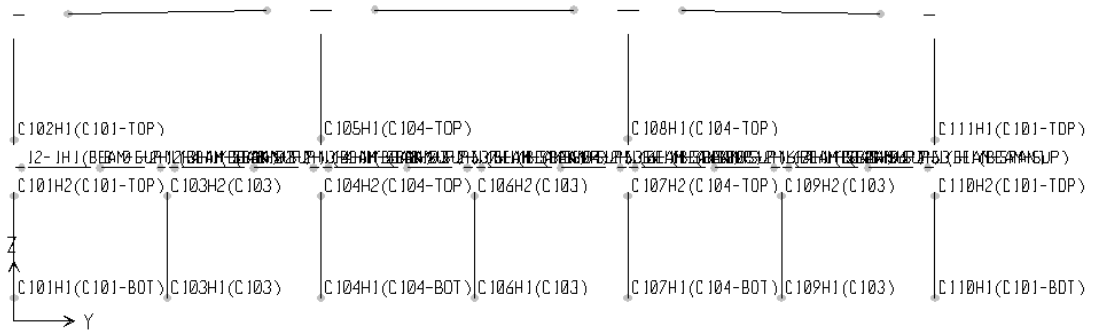


Figure 5.2. Plastic hinges assigned to sections in SAP2000.

Plastic hinge properties are determined by moment-curvature analyses utilizing an MS Excel [14] program developed using material models described in Section 2.1 and Section 2.6.(viii) of this study.

Moment-curvature analysis and diagram for pre-stressed beams are illustrated in the figures below. In the figures, c is the distance of the neutral axis from extreme compression fiber, F_c is the resultant compressive force, ϵ_{c0} and ϵ_{c1} are strains at extreme compression fiber and the top fiber of confined concrete, $F_{s1}, F_{s2}, \epsilon_{s1}, \epsilon_{s2}, f_{s1}$ and f_{s2} are forces, strains and stresses developed at top and bottom rebars, respectively, ϵ_{se} is the initial strain at pre-

stressing strands due to pre-stressing force, ϵ_{sa} is the strain at pre-stressing strands due to bending, ϵ_{ps} is the total strain at pre-stressing strands, F_{ps} is the resultant force at pre-stressing strands, ΣF is the total resultant force to check equilibrium, K is the curvature in 1/m and M is the moment in kNm units.

ϵ	F_c	ϵ_{c0}	ϵ_{c1}	F_{s1}	ϵ_{s1}	f_{s1}	F_{s2}	ϵ_{s2}	f_{s2}	F_{ps}	ϵ_{ps}	ϵ_{sa}	ϵ_{ps}	ΣF	K	M
26.400	-1329	0.00010	0.00008	-130.35	-8.1061E-05	-1.6212	262.86	0.0002598	5.197	1196.6	0.0059288	0.0002409	0.0061697	-0.22	0.0003788	1171.01
21.290	-2019	0.00030	0.00024	-369.11	-0.00022954	-4.5909	1050.7	0.0010387	20.773	1337.7	0.0059288	0.0009682	0.006897	0.028	0.0014091	2000.92
20.950	-2837	0.00060	0.00049	-734.54	-0.0004568	-9.136	2037.6	0.0021208	40.285	1533.5	0.0059288	0.0019776	0.0079063	-0.021	0.002864	3026.57
15.920	-2738	0.00100	0.00075	-1103	-0.00068593	-13.719	2080.3	0.0049673	41.129	1761.5	0.0059288	0.0046533	0.010582	0.751	0.0062814	3321.19
12.330	-2523	0.00150	0.00101	-1433.9	-0.00089173	-17.835	2153.4	0.0100572	42.575	1805	0.0059288	0.0094489	0.0153777	1.54	0.0121655	3466.32
10.520	-2364	0.00200	0.00124	-1687.5	-0.00104943	-20.989	2234.5	0.0160608	44.178	1818.1	0.0059288	0.0151103	0.021039	0.694	0.0190114	3568.98
9.910	-2294	0.00227	0.00135	-1808.5	-0.00112469	-22.494	2278.4	0.0194908	45.045	1821.7	0.0059288	0.0183455	0.0242743	-2.003	0.0229062	3616.94
6.890	-1194	0.00290	0.01260	-1279.2	-0.0007955	-15.91	2474.4	0.0370855	48.92	0	0.0059288	0.034981	0.0409097	0.8	0.04209	2251.74
6.290	-1539	0.00300	0.00109	-989.34	-0.00061526	-12.305	2523.3	0.04231	49.888	0	0.0059288	0.0399253	0.045854	-4.605	0.0476948	2306.58
6.170	-1522	0.00350	0.00123	-1067.2	-0.0006637	-13.274	2590.7	0.0503898	51.22	0	0.0059288	0.0475535	0.0534822	1.26	0.0567261	2367.89
6.140	-1516	0.00370	0.00129	-1104.6	-0.00068697	-13.739	2614.3	0.0535476	51.686	0	0.0059288	0.0505345	0.0564633	-6.556	0.0602606	2388.76
6.080	-1500	0.00400	0.00137	-1142.5	-0.00071053	-14.211	2648.1	0.0585	52.355	0	0.0059288	0.0552105	0.0611393	5.864	0.0657895	2419.82
6.19	-386		0.00140	-2251.2	-0.0014	-28	2636.7	0.0567735	52.13	0	0.0059288	0.0535772	0.0595059	-0.505	0.0639269	2382.19
5.80	-321		0.00150	-2412	-0.0015	-30	2730.7	0.0743333	53.988	0	0.0059288	0.0701667	0.0760954	-2.013	0.0833333	2465.68
5.56	-279		0.00155	-2492.4	-0.00155	-31	2772.3	0.0888667	54.81	0	0.0059288	0.0838987	0.0898275	0.658	0.099359	2502.44
5.44	-258		0.00157	-2524.6	-0.00157	-31.4	2781.5	0.0976453	54.991	0	0.0059288	0.0921939	0.0981226	-1.244	0.1090278	2510.2

Figure 5.3. Moment-curvature analysis results in MS Excel program.

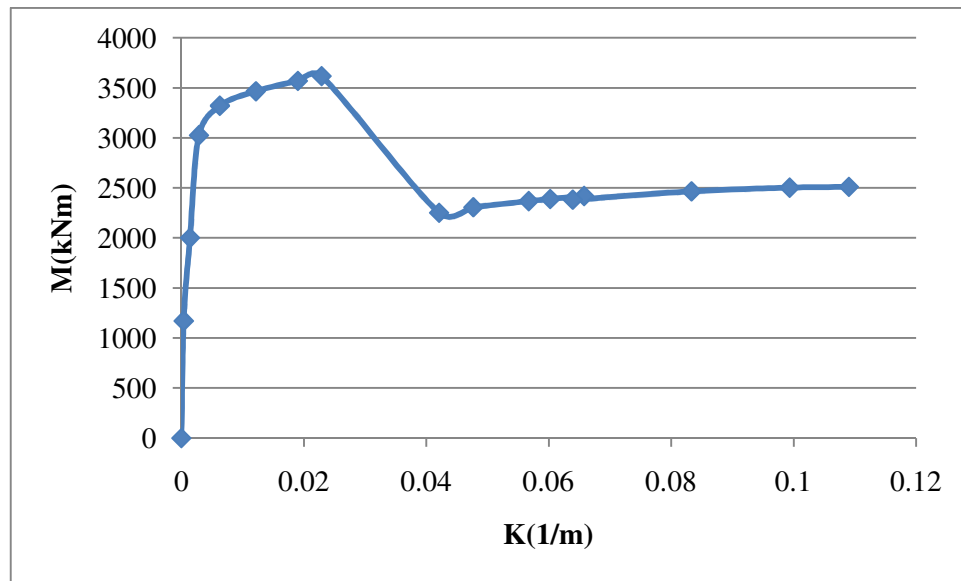
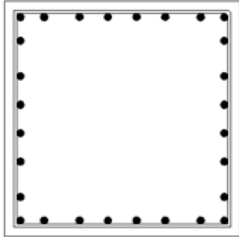


Figure 5.4. Moment-curvature relation for pre-stressed beams.

Note that the sudden drop in moment capacity of the section is due to rupture of pre-stressing strands, which have low ductility. Ultimate strain capacity of pre-stressing strands is 0.035. Moment-curvature analyses for column sections are also performed using the software developed by MS Excel. In Figure 5.6 below, ϵ_s , f_s and F_s are strains, stresses and forces at the rebars where the indices indicate the rebar row number. Rebars further than 4th row are not shown in the figure for clarity.



Input									
As1	53.24	cm ²	As5	10.64	cm ²	f_{yk}	3	kN/cm ²	
n1	5	cm	n5	54	cm	Σa_i^2	2705	cm ²	
As2	32.04	cm ²	As6	10.64	cm ²	b_s	92	cm	
n2	15	cm	n6	70	cm	h_s	92	cm	
As3	10.64	cm ²	As7	32.04	cm ²	s	10	cm	
n3	30	cm	n7	85	cm	As	213.12	cm ²	
As4	10.64	cm ²	As8	53.24	cm ²	P	2000	kN	
n4	42	cm	n8	95	cm				
ka	0.86849		fyk	42	kN/cm ²	B	100	cm	
fex	0.18968	kN/cm ²	Ea	20000	kN/cm ²	H	100	cm	
fey	0.18968	kN/cm ²	esu	0.1					
sec	0.00582		n_{conv}	4	cm	ecu	0.0187499		
foc	4.14593	kN/cm ²	px	0.0052	cm ³ /cm ³				
r	1.35158		py	0.0052	cm ³ /cm ³				
lc	1.38198								

Figure 5.5. Example input for C101-TOP section of Building 1.

n	ϵ_{s1}	ϵ_{s2}	ΣF	M	K	Fe	cs1	fs1	Fs1	cs2	fs2	Fs2	cs3	fs3	Fs3	cs4	fs4	Fs4
38.48	0.0005	0.000448	0.508	1794.6	0.001299	-2626.5	-0.000435	-8.701	-463.2	-0.000305	-6.102	-195.5	-0.00011	-2.2	-23.45	4.57E-05	0.9148	9.7331
35.45	0.0007	0.000621	0.958	2445	0.001975	-3206.8	-0.000601	-12.03	-640.2	-0.000404	-8.076	-258.8	-0.000108	-2.15	-22.9	0.000129	2.5867	27.523
33.85	0.0010	0.000882	0.785	3350	0.002954	-4007.7	-0.000852	-17.05	-907.5	-0.000557	-11.14	-356.8	-0.000114	-2.27	-24.2	0.000241	4.8154	51.235
29.60	0.0015	0.001297	0.952	4161.3	0.005068	-4616.1	-0.001247	-24.93	-1327	-0.00074	-14.8	-474.1	2.03E-05	0.405	4.3135	0.000628	12.568	133.72
25.06	0.0020	0.001681	0.672	4383.9	0.007981	-4627.3	-0.001601	-32.02	-1705	-0.000803	-16.06	-514.5	0.000394	7.885	83.897	0.001352	27.039	287.7
22.04	0.0025	0.002046	0.214	4511.6	0.011343	-4548.7	-0.001933	-38.66	-2058	-0.000799	-15.97	-511.7	0.000903	18.06	192.14	0.002264	42	446.88
20.35	0.0030	0.00241	-0.15	4603.9	0.014742	-4542	-0.002263	-42	-2236	-0.000789	-15.77	-505.4	0.001423	28.45	302.73	0.003192	42	446.88
19.67	0.0035	0.002788	1.831	4669.1	0.017794	-4657.1	-0.00261	-42	-2236	-0.000831	-16.62	-532.5	0.001838	36.76	391.14	0.003973	42	446.88
19.47	0.0037	0.00294	2.762	4694.2	0.019004	-4701.5	-0.00275	-42	-2236	-0.000849	-16.99	-544.3	0.002001	40.02	425.83	0.004282	42	446.88
19.28	0.0039	0.003091	-2.09	4716	0.020228	-4739.5	-0.002889	-42	-2236	-0.000866	-17.32	-554.8	0.002168	42	446.88	0.004596	42	446.88
19.15	0.0040	0.003164	2.677	4727.1	0.020888	-4746.3	-0.002956	-42	-2236	-0.000867	-17.34	-555.5	0.002266	42	446.88	0.004773	42	446.88
21.96		0.0032	0.67	4576.2	0.017817	-4299.6	-0.003022	-42	-2236	-0.00124	-24.8	-794.6	0.001433	28.65	304.84	0.003571	42	446.88
20.84		0.0040	0.01	4694.9	0.023753	-4455.2	-0.003762	-42	-2236	-0.001387	-27.74	-888.9	0.002176	42	446.88	0.005026	42	446.88
19.75		0.0050	1.58	4814.9	0.031746	-4522.3	-0.004683	-42	-2236	-0.001508	-30.16	-966.3	0.003254	42	446.88	0.007063	42	446.88
19.12		0.0060	-0.31	4920.4	0.039683	-4581.1	-0.005603	-42	-2236	-0.001635	-32.7	-1048	0.004317	42	446.88	0.009079	42.303	450.11
18.72		0.0070	-1.54	5014.8	0.047554	-4626	-0.006524	-42	-2236	-0.001769	-35.38	-1134	0.005364	42	446.88	0.011071	42.853	455.96
18.39		0.0127	0.02	5360.2	0.088256	-4864.5	-0.011817	-43.06	-2292	-0.002992	-42	-1346	0.010246	42.63	453.55	0.020837	45.375	482.79
18.46		0.0135	0.48	5388	0.093361	-4898.2	-0.012566	-43.26	-2303	-0.00323	-42	-1346	0.010774	42.77	455.1	0.021977	45.65	485.72
18.66		0.0158	1.14	5449.9	0.107776	-4972.5	-0.014722	-43.83	-2334	-0.003945	-42	-1346	0.012222	43.17	459.28	0.025155	46.396	493.65
18.75		0.0168	-2.43	5468.5	0.113898	-4998.2	-0.015661	-44.07	-2347	-0.004271	-42	-1346	0.012814	43.32	460.98	0.026481	46.698	496.87
18.85		0.0183	1.04	5490.6	0.123232	-5018.9	-0.017068	-44.44	-2366	-0.004744	-42	-1346	0.01374	43.57	463.6	0.028528	47.154	501.72
18.89		0.0188	-1.98	5495.1	0.126259	-5026.7	-0.017537	-44.56	-2372	-0.004911	-42	-1346	0.014027	43.65	464.41	0.029179	47.296	503.23

Figure 5.6. Moment-curvature analysis results for Building 1 C101-TOP section.

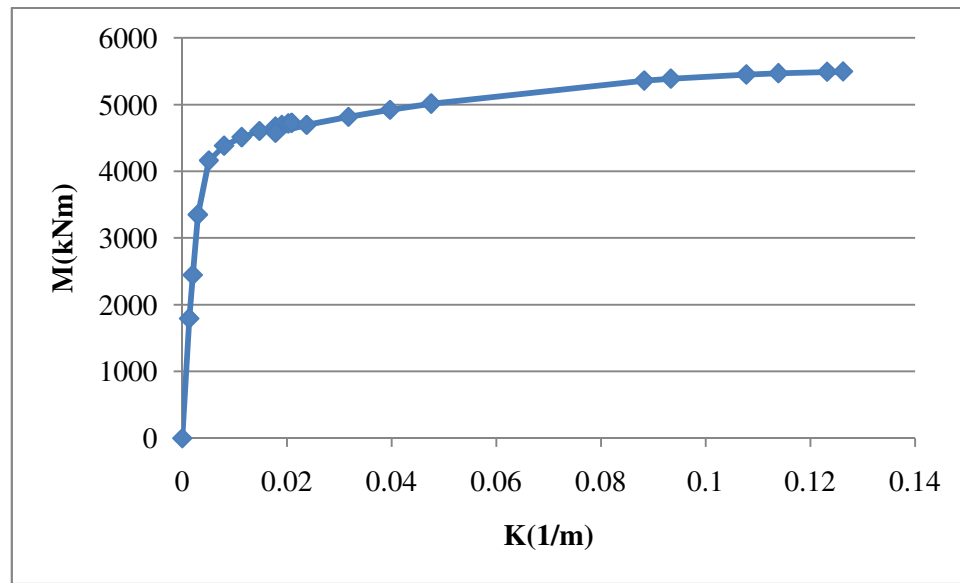


Figure 5.7. Moment-curvature diagram for Building 1 C101-TOP section (N=2000kN).

Plastic hinge properties are defined in SAP2000 using moment-curvature analyses performed by the software described above. Plastic hinge properties defined in SAP2000 for Building 1 C101-TOP section are shown as an example in Figure 5.8.

Plastic hinge properties are defined in SAP2000 in terms of five points A through E. Point A represents initial stage, Point B represents yielding, Point C represents the strength of the section. Beyond Point D, the member responds with reduced strength to Point E. Point E is the point at which total crushing occurs.

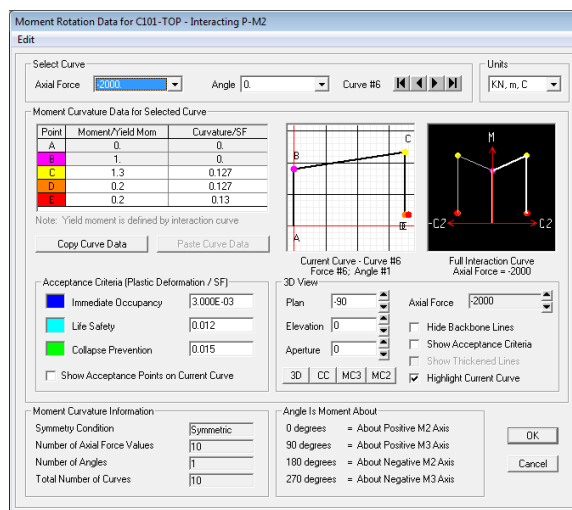


Figure 5.8. Plastic hinge properties defined for C101-TOP section in SAP2000.

Moment-Interaction diagrams of the sections are obtained using XTRACT sectional analysis software and defined in SAP2000. (Figure 5.9)

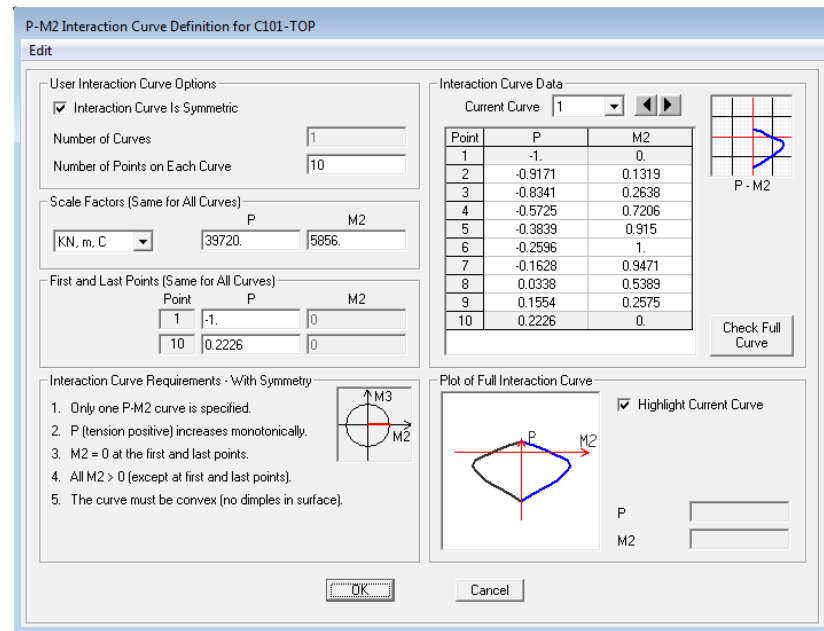


Figure 5.9. Moment-interaction diagram defined in SAP2000.

After defining hinge properties, statical nonlinear pushover analysis is performed by SAP2000 and pushover curve is obtained. The structure is then ‘pushed’ to the displacement demanded by earthquake and plastic hinge rotations are obtained. These rotations are then plugged into the moment-curvature analyses and strain values for concrete and strain are compared with limit values described in Section 2.6 of this study.

5.2 Nonlinear Static Performance Analysis for Building 1

Plastic hinges are assigned to potential plastic hinge regions in SAP2000 as described above. Modal analysis results, to be used in distribution of lateral forces in accordance with TSC2007 Section 7.6.5.3, are shown in Table 5.1 below.

Table 5.1. Modal analysis results for Building 1.

Mode Number	T (sec)	$M_i/\Sigma m$	Φ_1	Φ_2
1	1.201	0.758	1.60	5.26
2	0.689	0.242	-1.79	4.69

In Table 5.1;

T : Natural vibration period of the frame

$M_i/\Sigma m$: Ratio of effective mass of i^{th} mode to total mass of the frame

Φ_1, Φ_2 : Mode shape vectors for first and second storey, respectively

Note that the ratio of effective mass of first mode to total mass is larger than 0.70, so *Incremental Equivalent Seismic Force Method* is applicable according to TSC2007 Section 7.6.5.2.

Lateral loads are distributed to the stories proportional to the product of their mass and mode shape vector.

$$\frac{F_1}{F_2} = \frac{m_1 * \Phi_1}{m_2 * \Phi_2} = \frac{1756.7 * 1.60}{193.3 * 5.26} = 2.764$$

Lateral load distribution is defined in SAP2000 and pushover analysis is performed. Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below.

$$M_{x1} = \frac{L_{x1}^2}{M_1} = \frac{(1756.7kNs^2/m * 0.0160m + 193.3kNs^2/m * 0.0526m)^2}{1756.7kNs^2/m * (0.0160m)^2 + 193.3kNs^2/m * (0.0526m)^2} = 1488kNs^2/m$$

$$\Phi_{xN1} \Gamma_{x1} = 0.0526m * \frac{38.27kNs^2}{0.985kNs^2m} = 2.045$$

Table 5.2. Pushover analysis results for Building 1.

Step(i)	$u_{xN1}^{(i)}$ (m)	$V_{x1}^{(i)}$ (kN)	$d_1^{(i)}$ (m)	$a_1^{(i)}$ (m/s ²)
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.060323	1222.1	0.029498	0.821329
2	0.120323	2444.3	0.058838	1.642657
3	0.193448	3854.3	0.094596	2.590263
4	0.257937	4631.5	0.126131	3.112536
5	0.366335	4849.1	0.179137	3.258826
6	0.426335	4897.5	0.208477	3.291302
7	0.526961	4959.0	0.257683	3.332656
8	0.624489	4970.2	0.305374	3.340181
9	0.684489	4963.4	0.334713	3.335591
10	0.744489	4956.5	0.364053	3.330997
11	0.804489	4949.7	0.393393	3.326401
12	0.864489	4942.8	0.422733	3.3218
13	0.924489	4936.0	0.452073	3.317198
14	0.984489	4929.1	0.481413	3.312591
15	1.044489	4922.3	0.510753	3.307982
16	1.104489	4915.4	0.540092	3.303369
17	1.183157	4876.2	0.578561	3.277014
18	1.254526	4777.4	0.61346	3.210603
19	1.319244	4623.9	0.645107	3.107444
20	1.39992	4475.3	0.684557	3.007611
21	1.463983	4138.7	0.715884	2.781369
22	1.544779	3885.4	0.755393	2.611161
23	1.607621	3694.1	0.786123	2.48261
24	1.674238	3485.8	0.818698	2.342596
25	1.734853	3360.5	0.848339	2.258413

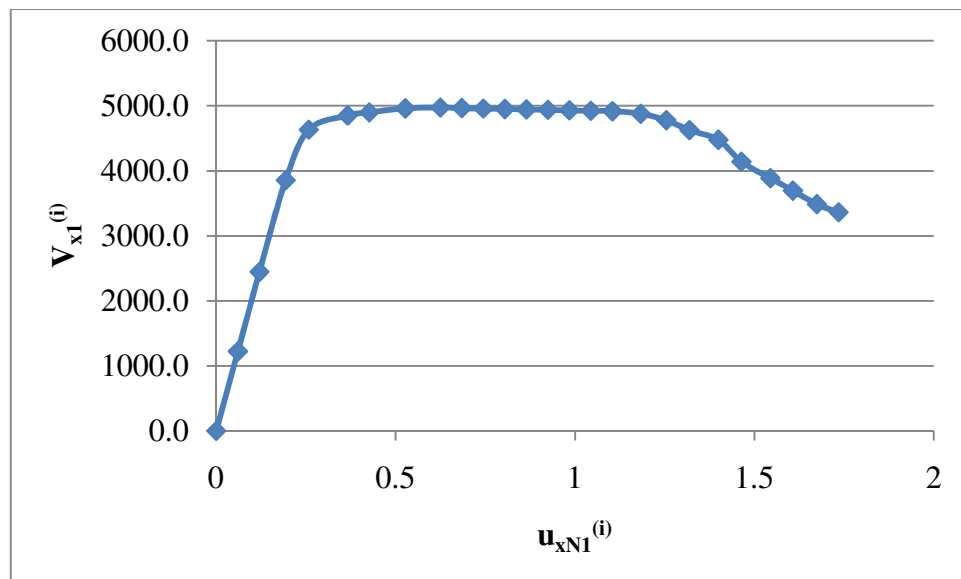


Figure 5.10. Pushover curve for Building 1.

Modal displacement demand is determined according to TSC2007 Appendix 7C as described in Section 2.7.(xiv) of this study.

$T_1 = 1.201s > T_B = 0.60s$, therefore $C_{R1} = 1.0$

$$S_{di1} = C_{R1} S_{de1} = 1.0 * \frac{S_{ae1}}{(\omega_1^{(1)})^2} = \frac{S_{ae1}}{(2\pi/1.201)^2} = \frac{(9.81)(2.5 * (0.60/1.201)^{0.8})(0.40)}{27.37} = 0.206m$$

In the figure below, adapted pushover curve and modal spectrum curve can be observed.

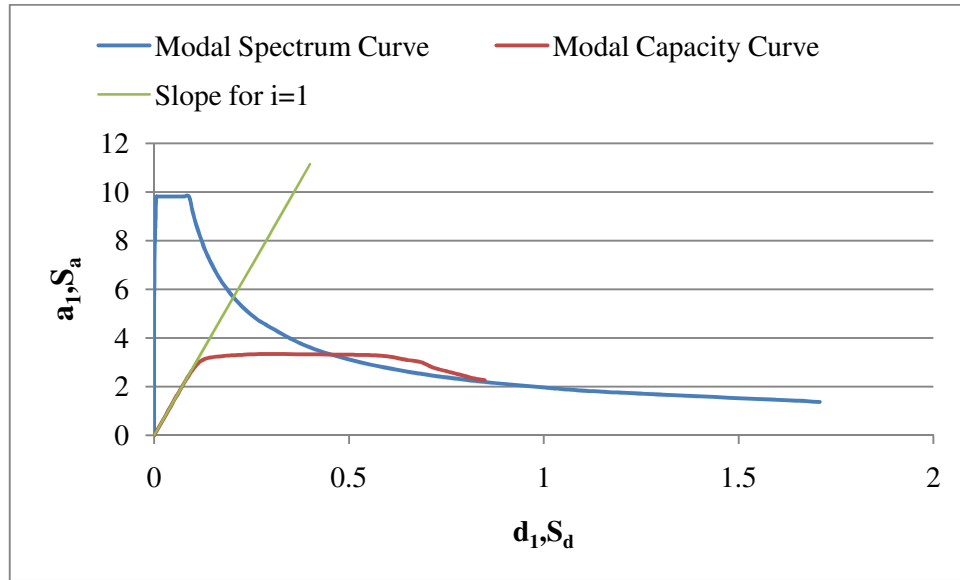


Figure 5.11. Modal spectrum and capacity curves for Building 1.

Equation 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0526m * 38.853/m * 0.206m = 0.420m$$

The tip of the frame is ‘pushed’ to 0.420m in SAP2000. Plastic hinges formed in the frame at 0.420m displacement demand are shown in Figure 5.12.

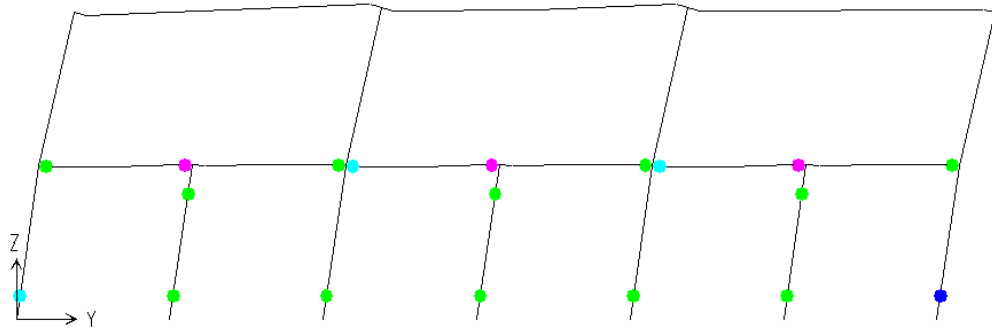


Figure 5.12. Plastic hinge formations for Building 1 at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.3. Plastic hinge curvature demands and performance levels for Building 1.

Hinge	Θ_p	Φ_p	Φ_v	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0104	0.0208	0.0029	0.0237	0.0199	0.0023	0.0014	VDL
B101-R	-0.0012	-0.0024	-0.0029	-0.0053	0.0043	0.0009	0.0007	MDL
B102-R	-0.0089	-0.0178	-0.0029	-0.0207	0.0179	0.0024	0.0015	VDL
B103-L	0.0061	0.0122	0.0029	0.0151	0.0127	0.0016	0.0010	VDL
B103-R	-0.0010	-0.0020	-0.0029	-0.0049	0.0040	0.0009	0.0007	MDL
B104-R	-0.0090	-0.0180	-0.0029	-0.0209	0.0179	0.0024	0.0015	VDL
B105-L	0.0062	0.0124	0.0029	0.0153	0.0127	0.0016	0.0010	VDL
B105-R	-0.0006	-0.0012	-0.0029	-0.0041	0.0034	0.0008	0.0006	MDL
B106-R	-0.0141	-0.0282	-0.0029	-0.0311	0.0273	0.0033	0.0020	VDL
C101-B	0.0072	0.0144	0.0060	0.0204	0.0160	0.0034	0.0026	VDL
C102-B	0.0101	0.0202	0.0047	0.0249	0.0180	>0.004	0.0045	VDL
C102-T	-0.0088	-0.0176	-0.0047	-0.0223	0.0170	>0.004	0.0043	VDL
C103-B	0.0096	0.0192	0.0051	0.0243	0.0180	>0.004	0.0042	VDL
C104-B	0.0103	0.0206	0.0047	0.0253	0.0180	>0.004	0.0045	VDL
C104-T	-0.0085	-0.0170	-0.0047	-0.0217	0.0160	>0.004	0.0042	VDL
C105-B	0.0096	0.0192	0.0051	0.0243	0.0180	>0.004	0.0042	VDL
C106-B	0.0102	0.0204	0.0047	0.0251	0.0180	>0.004	0.0045	VDL
C106-T	-0.0090	-0.0180	-0.0047	-0.0227	0.0160	>0.004	0.0042	VDL
C107-B	0.0059	0.0118	0.0052	0.0170	0.0127	>0.004	0.0032	VDL

5.3 Nonlinear Static Performance Analysis for Building 1A

Modal analysis results, to be used in distribution of lateral forces in accordance with TSC2007 Section 7.6.5.3, are shown in Table 5.4 below.

Table 5.4. Modal analysis results for Building 1A.

Mode Number	T (sec)	$M_i/\Sigma m$	Φ_1	Φ_2
1	0.762	0.721	1.39	4.74
2	0.338	0.278	-1.69	3.58

In the table above;

T : Natural vibration period of the frame

$M_i/\Sigma m$: Ratio of effective mass of i^{th} mode to total mass of the frame

Φ_1, Φ_2 : Mode shape vectors for first and second storey, respectively

Note that the ratio of effective mass of first mode to total mass is larger than 0.70, so *Incremental Equivalent Seismic Force Method* is applicable according to TSC2007 Section 7.6.5.2.

Lateral loads are distributed to the stories proportional to the product of their mass and mode shape vector.

$$\frac{F_1}{F_2} = \frac{m_1 * \Phi_1}{m_2 * \Phi_2} = \frac{1975.3 * 1.39}{272.8 * 4.74} = 2.101$$

Lateral load distribution is defined in SAP2000 and pushover analysis is performed.

Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below.

$$M_{x1} = \frac{L_{x1}^2}{M_1} = \frac{(1975.3kNs^2/m * 0.0139m + 272.8kNs^2/m * 0.0474m)^2}{1975.3kNs^2/m * (0.0139m)^2 + 272.8kNs^2/m * (0.0474m)^2} = 1640kNs^2/m$$

$$\Phi_{xN1} \Gamma_{x1} = 0.0474m * \frac{40.39kNs^2}{0.995kNs^2m} = 1.925$$

Table 5.5. Pushover analysis results for Building 1A.

Step(i)	$u_{xN1}^{(i)}$ (m)	$V_{x1}^{(i)}$ (kN)	$d_1^{(i)}$ (m)	$a_1^{(i)}$ (m/s ²)
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.044179	3264.2	0.02295	1.990388
2	0.115492	6556.9	0.059996	3.998114
3	0.163231	7571.2	0.084795	4.616613
4	0.168597	7615.9	0.087583	4.643817
5	0.318597	7770.9	0.165505	4.73834
6	0.375255	7820.1	0.194938	4.768329
7	0.413804	7823.1	0.214963	4.77021
8	0.429191	7819.5	0.222956	4.767984
9	0.659252	7620.0	0.342469	4.646334
10	0.889297	7420.4	0.461972	4.524638
11	0.950712	7367.1	0.493876	4.492139
12	0.954549	7338.7	0.49587	4.47479
13	0.970043	7131.3	0.503918	4.348342

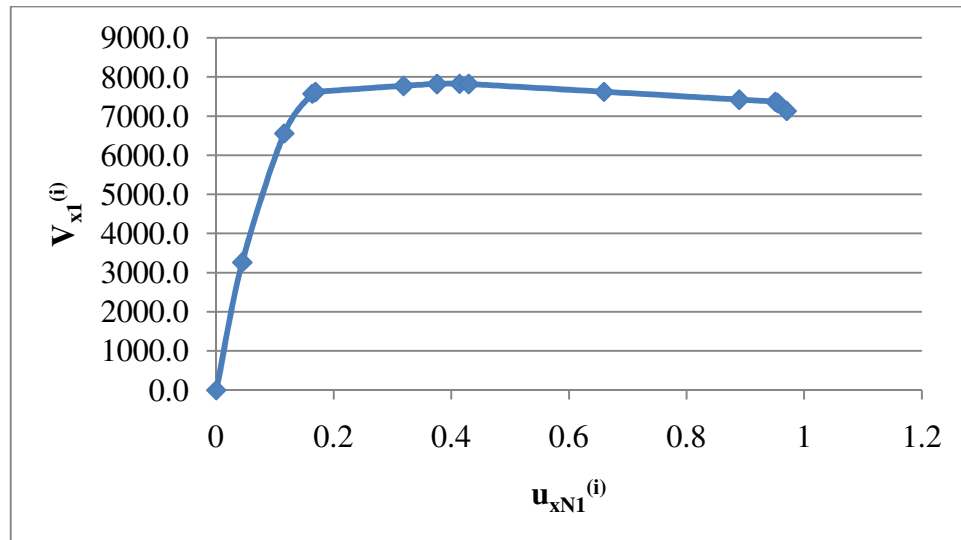


Figure 5.13. Pushover curve for Building 1A.

Modal displacement demand is determined according to TSC2007 Appendix 7C as described in Section 2.7.(xiv) of this study.

$T_1 = 0.762s > T_B = 0.60s$, therefore $C_{R1} = 1.0$

$$S_{di1} = C_{R1} S_{de1} = 1.0 * \frac{S_{ae1}}{(\omega_1^{(1)})^2} = \frac{S_{ae1}}{(2\pi/0.762)^2} = \frac{(9.81)(2.5 * (0.60/0.762)^{0.8})(0.40)}{67.99} = 0.119m$$

In the figure below, adapted pushover curve and modal spectrum curve can be observed.

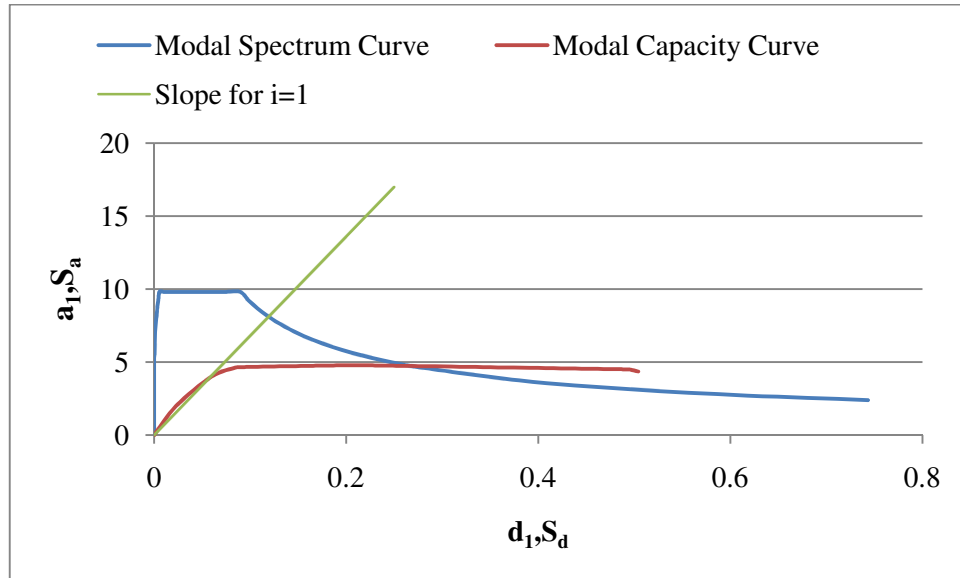


Figure 5.14. Modal spectrum and capacity curves for Building 1A.

Equation 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0474m * 40.593/m * 0.119m = 0.229m$$

The tip of the frame is ‘pushed’ to 0.229m in SAP2000. Plastic hinges formed in the frame at 0.229m displacement demand are shown in the figure below.

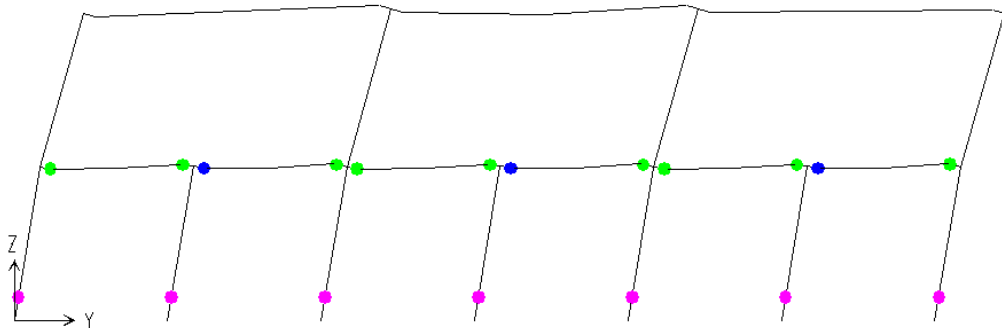


Figure 5.15. Plastic hinge formations for Building 1A at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.6. Plastic hinge curvature demands and performance levels for Building 1A.

Hinge	Θ_p	Φ_p	Φ_y	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0087	0.0174	0.0039	0.0213	0.0186	0.0018	0.0009	VDL
B101-R	-0.0098	-0.0196	-0.0035	-0.0231	0.0198	0.0024	0.0015	VDL
B102-L	0.0052	0.0104	0.0039	0.0143	0.0122	0.0014	0.0008	VDL
B102-R	-0.0122	-0.0244	-0.0035	-0.0279	0.0239	0.0028	0.0017	VDL
B103-L	0.0082	0.0164	0.0039	0.0203	0.0179	0.0018	0.0009	VDL
B103-R	-0.0098	-0.0196	-0.0035	-0.0231	0.0198	0.0024	0.0015	VDL
B104-L	0.0052	0.0104	0.0039	0.0143	0.0122	0.0014	0.0008	VDL
B104-R	-0.0122	-0.0244	-0.0035	-0.0279	0.0239	0.0028	0.0017	VDL
B105-L	0.0082	0.0164	0.0039	0.0203	0.0179	0.0018	0.0009	VDL
B105-R	-0.0098	-0.0196	-0.0035	-0.0231	0.0198	0.0024	0.0015	VDL
B106-L	0.0051	0.0102	0.0039	0.0141	0.0122	0.0014	0.0008	VDL
B106-R	-0.0129	-0.0258	-0.0035	-0.0293	0.0251	0.0029	0.0018	VDL
C101-B	0.0032	0.0040	0.0029	0.0069	0.0088	0.0017	0.0014	MDL
C102-B	0.0050	0.0063	0.0027	0.0090	0.0118	0.0021	0.0017	VDL
C103-B	0.0032	0.0040	0.0025	0.0065	0.0081	0.0018	0.0015	MDL
C104-B	0.0050	0.0063	0.0027	0.0090	0.0118	0.0021	0.0017	VDL
C105-B	0.0032	0.0040	0.0025	0.0065	0.0081	0.0018	0.0015	MDL
C106-B	0.0050	0.0063	0.0027	0.0090	0.0118	0.0021	0.0017	VDL
C107-B	0.0030	0.0038	0.0027	0.0065	0.0081	0.0018	0.0015	MDL

5.4 Nonlinear Static Performance Analysis for Building 2

Modal analysis results, to be used in distribution of lateral forces in accordance with TSC2007 Section 7.6.5.3, are shown in Table 5.7 below.

Table 5.7. Modal analysis results for Building 2.

Mode Number	T (sec)	$M_i/\Sigma m$	Φ_1	Φ_2
1	0.882	0.786	1.72	5.39
2	0.529	0.214	-1.74	5.26

In Table 5.7;

T : Natural vibration period of the frame

$M_i/\Sigma m$: Ratio of effective mass of i^{th} mode to total mass of the frame

Φ_1, Φ_2 : Mode shape vectors for first and second storey, respectively

Note that the ratio of effective mass of first mode to total mass is larger than 0.70, so *Incremental Equivalent Seismic Force Method* is applicable according to TSC2007 Section 7.6.5.2.

Lateral loads are distributed to the stories proportional to the product of their mass and mode shape vector.

$$\frac{F_1}{F_2} = \frac{m_1 * \Phi_1}{m_2 * \Phi_2} = \frac{1687.4 * 1.72}{168.1 * 5.39} = 3.203$$

Lateral load distribution is defined in SAP2000 and pushover analysis is performed. Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below.

$$M_{x1} = \frac{L_{x1}^2}{M_1} = \frac{(1687.4kNs^2/m * 0.0172m + 168.1kNs^2/m * 0.0539m)^2}{1687.4kNs^2/m * (0.0172m)^2 + 168.1kNs^2/m * (0.0539m)^2} = 1468kNs^2/m$$

$$\Phi_{xN1} \Gamma_{x1} = 0.0539m * \frac{38.08kNs^2}{0.988kNs^2m} = 2.077$$

Table 5.8. Pushover analysis results for Building 2.

Step(i)	$u_{xN1}^{(i)} (m)$	$V_{x1}^{(i)} (kN)$	$d_1^{(i)} (m)$	$a_1^{(i)} (m/s^2)$
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.124432	4547.6	0.059909	3.097799
2	0.142687	5121.7	0.068699	3.48892
3	0.15713	5370.5	0.075652	3.658373
4	0.173259	5500.3	0.083418	3.746824
5	0.241654	5808.0	0.116348	3.956409
6	0.377538	5872.2	0.181771	4.000129
7	0.679241	4761.2	0.32703	3.243321

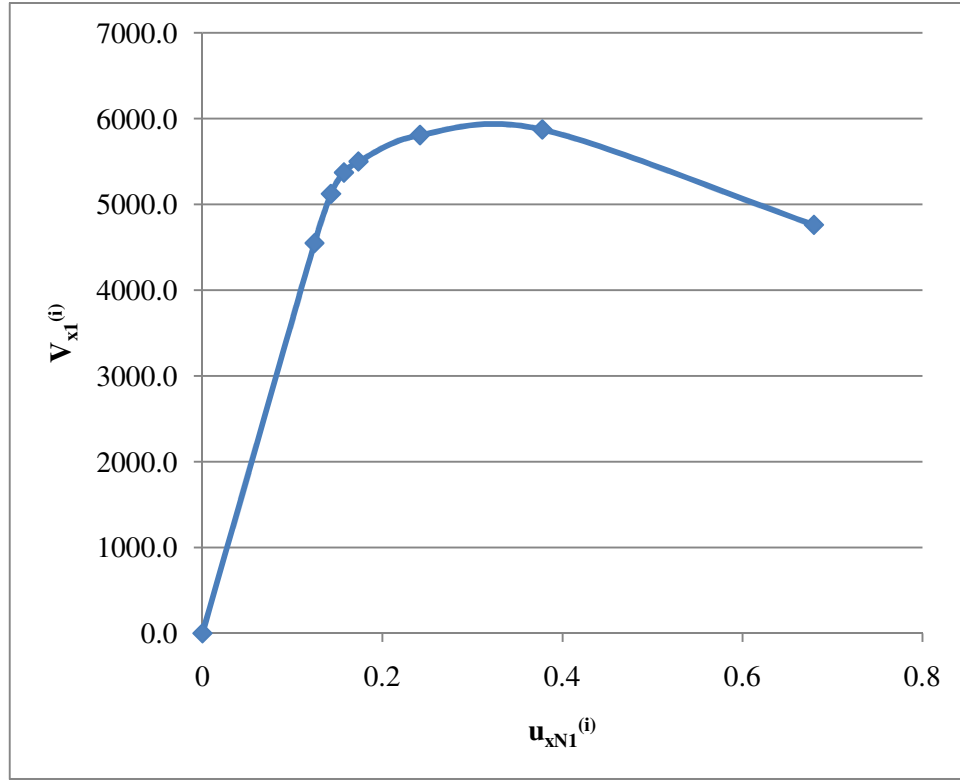


Figure 5.16. Pushover curve for Building 2.

Modal displacement demand is determined according to TSC2007 Appendix 7C as described in Section 2.7.(xiv) of this study.

$T_1 = 0.882s > T_B = 0.60s$, therefore $C_{R1} = 1.0$

$$S_{di1} = C_{R1} S_{de1} = 1.0 * \frac{S_{ae1}}{(\omega_1^{(1)})^2} = \frac{S_{ae1}}{(2\pi/0.882)^2} = \frac{(9.81)(2.5 * (0.60/0.882)^{0.8})(0.40)}{50.75} = 0.142m$$

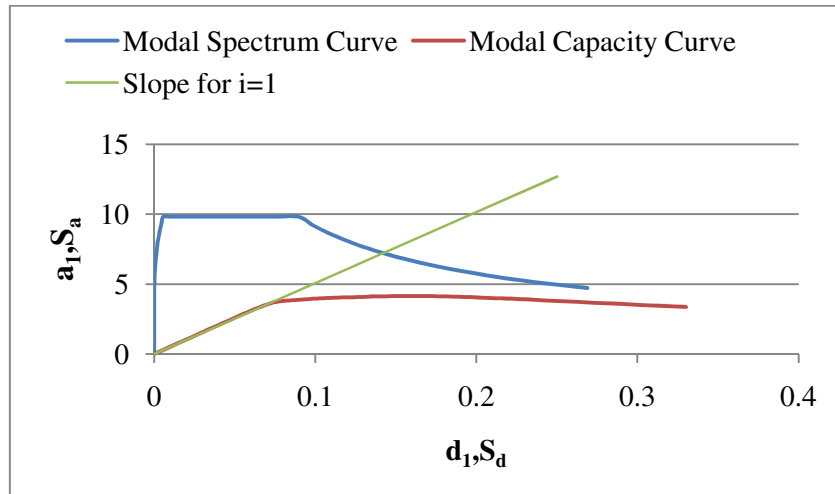


Figure 5.17. Modal spectrum and capacity curves for Building 2.

Equations 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0539m * 38.543/m * 0.142m = 0.295m$$

The tip of the frame is ‘pushed’ to 0.295m in SAP2000. Plastic hinges formed in the frame at 0.295m displacement demand are shown in the figure below.

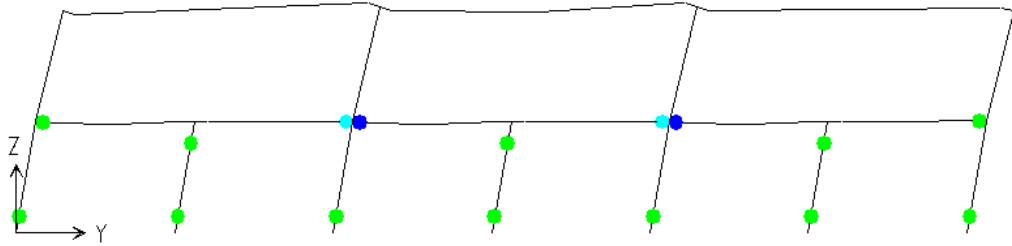


Figure 5.18. Plastic hinge formations for Building 2 at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.9. Plastic hinge curvature demands and performance levels for Building 2.

Hinge	Θ_p	Φ_p	Φ_v	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0105	0.0210	0.0031	0.0241	0.0198	0.0021	0.0012	VDL
B102-R	-0.0061	-0.0132	-0.0028	-0.0160	0.0136	0.0020	0.0014	VDL
B103-L	0.0040	0.0080	0.0031	0.0111	0.0089	0.0013	0.0009	MDL
B104-R	-0.0062	-0.0134	-0.0028	-0.0162	0.0136	0.0020	0.0014	VDL
B105-L	0.0040	0.0080	0.0031	0.0111	0.0089	0.0013	0.0009	MDL
B106-R	-0.0129	-0.0258	-0.0028	-0.0286	0.0256	0.0031	0.0019	VDL
C101-B	0.0104	0.0245	0.0061	0.0306	0.0188	>0.004	0.0043	VDL
C102-B	0.0123	0.0289	0.0070	0.0359	0.0206	>0.004	0.0066	VDL
C102-T	-0.0122	-0.0287	0.0070	0.0357	0.0206	>0.004	0.0066	VDL
C103-B	0.0139	0.0327	0.0049	0.0376	0.0220	>0.004	0.0064	VDL
C104-B	0.0122	0.0287	0.0070	0.0357	0.0206	>0.004	0.0066	VDL
C104-T	-0.0123	0.0289	0.0070	0.0359	0.0206	>0.004	0.0066	VDL
C105-B	0.0139	0.0327	0.0049	0.0376	0.0220	>0.004	0.0064	VDL
C106-B	0.0123	0.0289	0.0070	0.0359	0.0206	>0.004	0.0066	VDL
C106-T	-0.0122	0.0287	0.0070	0.0357	0.0206	>0.004	0.0066	VDL
C107-B	0.0095	0.0223	0.0051	0.0274	0.0164	>0.004	0.0047	VDL

5.5 Nonlinear Static Performance Analysis for Building 2A

Modal analysis results, to be used in distribution of lateral forces in accordance with TSC2007 Section 7.6.5.3, are shown in Table 5.10 below.

Table 5.10. Modal analysis results for Building 2A.

Mode Number	T (sec)	$M_i/\Sigma m$	Φ_1	Φ_2
1	0.502	0.760	1.59	5.14
2	0.247	0.238	-1.77	4.46

In the table above;

T : Natural vibration period of the frame

$M_i/\Sigma m$: Ratio of effective mass of i^{th} mode to total mass of the frame

Φ_1, Φ_2 : Mode shape vectors for first and second storey, respectively

Note that the ratio of effective mass of first mode to total mass is larger than 0.70, so *Incremental Equivalent Seismic Force Method* is applicable according to TSC2007 Section 7.6.5.2.

Lateral loads are distributed to the stories proportional to the product of their mass and mode shape vector.

$$\frac{F_1}{F_2} = \frac{m_1 * \Phi_1}{m_2 * \Phi_2} = \frac{1764.6 * 1.59}{201.8 * 5.14} = 2.705$$

Lateral load distribution is defined in SAP2000 and pushover analysis is performed. Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below.

$$M_{x1} = \frac{L_{x1}^2}{M_1} = \frac{(1746.6kNs^2/m * 0.0159m + 201.8kNs^2/m * 0.0514m)^2}{1746.6kNs^2/m * (0.0159m)^2 + 201.8kNs^2/m * (0.0514m)^2} = 1492kNs^2/m$$

$$\Phi_{xv1} \Gamma_{x1} = 0.0514m * \frac{38.14kNs^2}{0.975kNs^2m} = 2.011$$

Table 5.11. Pushover analysis results for Building 2A.

Step(i)	$u_{xN1}^{(i)}$ (m)	$V_{x1}^{(i)}$ (kN)	$d_1^{(i)}$ (m)	$a_1^{(i)}$ (m/s ²)
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.034221	4066.2	0.017017	2.725359
2	0.064255	6499.9	0.031952	4.356488
3	0.088783	7487.4	0.044149	5.018345
4	0.093705	7583.5	0.046596	5.082742
5	0.108709	7752.6	0.054057	5.196137
6	0.109836	7758.6	0.054618	5.200118
7	0.276102	8121.8	0.137296	5.443597
8	0.31279	8134.6	0.15554	5.452133
9	0.660253	7603.6	0.328321	5.096275
10	0.697336	7547.0	0.346761	5.058292
11	0.709105	7337.4	0.352613	4.917802

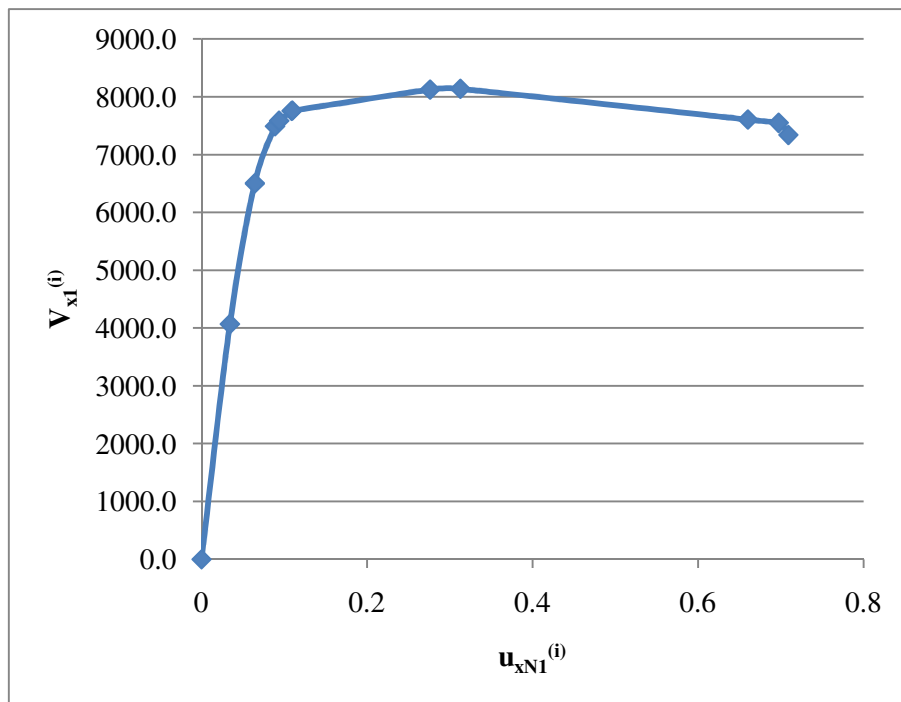


Figure 5.19. Pushover curve for Building 2A.

Modal displacement demand is determined according to TSC2007 Appendix 7C as described in Section 2.7.(xiv) of this study.

$$T_1 = 0.502s < T_B = 0.60s$$

According to TSC2007 Appendix 7C, sequential analysis method is used to determine C_{R1} .

Table 5.12. Determination of C_{R1} for Building 2A.

Step	C_{R1}	S_{di1}	a_{v1}	R_{v1}
1	1.0	0.0626	2.73	3.593
2	1.141	0.0715	1.01	9.713
3	1.175	0.0736	-	-

Note that S_{di1} 's at 2nd and 3rd steps are reasonably close to each other.

$$S_{di1} = C_{R1} S_{de1} = 1.175 * \frac{S_{ae1}}{(\omega_1^{(1)})^2} = \frac{S_{ae1}}{(2\pi/0.502)^2} = \frac{(9.81)(2.5^*)(0.40)}{156.66} = 0.0736m$$

In the figure below, adapted pushover curve and modal spectrum curve can be observed.

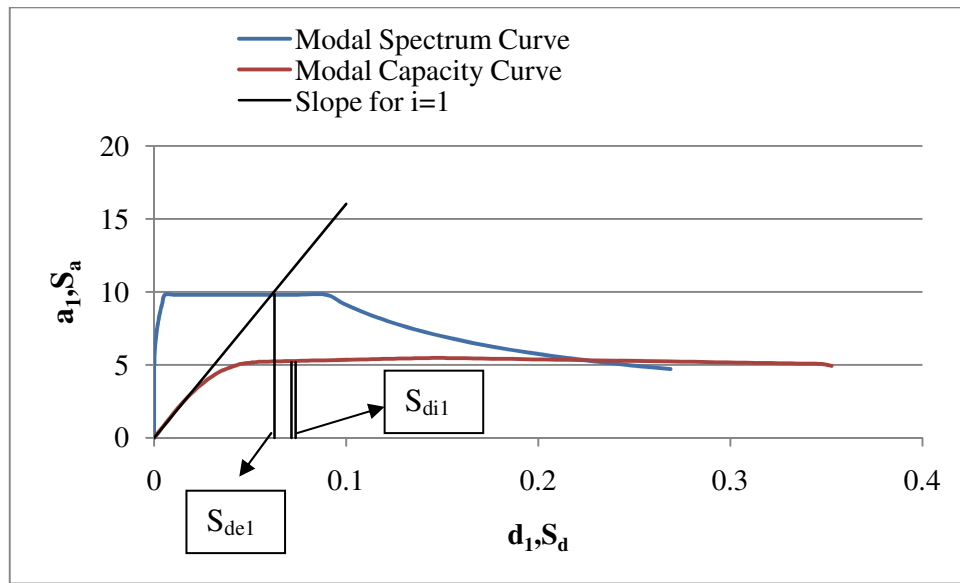


Figure 5.20. Modal spectrum and capacity curves for Building 2A.

Equation 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0514m * 39.118/m * 0.0736m = 0.148m$$

The tip of the frame is ‘pushed’ to 0.148m in SAP2000. Plastic hinges formed in the frame at 0.148m displacement demand are shown in Figure 5.21.

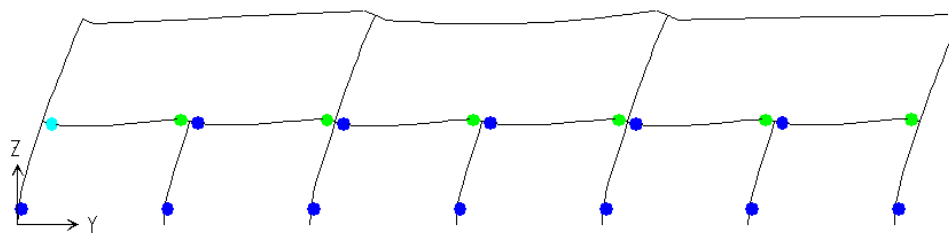


Figure 5.21. Plastic hinge formations for Building 2A at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.13. Plastic hinge curvature demands and performance levels for Building 2A.

Hinge	Θ_p	Φ_p	Φ_y	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0066	0.0132	0.0039	0.0171	0.0148	0.0016	0.0010	VDL
B101-R	-0.0081	-0.0162	-0.0035	-0.0197	0.0169	0.0021	0.0013	VDL
B102-L	0.0030	0.0060	0.0039	0.0099	0.0084	0.0012	0.0008	MDL
B102-R	-0.0103	-0.0206	-0.0035	-0.0241	0.0210	0.0025	0.0015	VDL
B103-L	0.0058	0.0116	0.0039	0.0155	0.0130	0.0015	0.0009	VDL
B103-R	-0.0081	-0.0162	-0.0035	-0.0197	0.0169	0.0021	0.0013	VDL
B104-L	0.0030	0.0060	0.0039	0.0099	0.0084	0.0012	0.0008	MDL
B104-R	-0.0103	-0.0206	-0.0035	-0.0241	0.0210	0.0025	0.0015	VDL
B105-L	0.0058	0.0116	0.0039	0.0155	0.0130	0.0015	0.0009	VDL
B105-R	-0.0081	-0.0162	-0.0035	-0.0197	0.0169	0.0021	0.0013	VDL
B106-L	0.0029	0.0058	0.0039	0.0097	0.0084	0.0012	0.0008	VDL
B106-R	-0.0114	-0.0228	-0.0035	-0.0263	0.0229	0.0027	0.0016	VDL
C101-B	0.0041	0.0063	0.0033	0.0096	0.0133	0.0020	0.0016	VDL
C102-B	0.0059	0.0091	0.0027	0.0118	0.0159	0.0025	0.0020	VDL
C103-B	0.0041	0.0063	0.0025	0.0088	0.0109	0.0022	0.0018	VDL
C104-B	0.0059	0.0091	0.0027	0.0118	0.0159	0.0025	0.0020	VDL
C105-B	0.0041	0.0063	0.0025	0.0088	0.0109	0.0022	0.0018	VDL
C106-B	0.0059	0.0091	0.0027	0.0118	0.0159	0.0025	0.0020	VDL
C107-B	0.0038	0.0058	0.0027	0.0085	0.0113	0.0021	0.0017	VDL

5.6 Nonlinear Static Performance Analysis for Building 1B

In light of the results obtained from performance analysis of Buildings 1, 1A, 2 and 2A, an alternative design strategy is developed. According to this strategy, the short columns (C102,C104,C106) are provided with longitudinal reinforcement identical to top sections of middle columns (C103, C105). Therefore, short columns of Building 1B are provided with 16Φ32+24Φ26.

Lateral load distribution is defined in SAP2000 and pushover analysis is performed. Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below. Since modal analysis results are obviously same with Building 1, they are not presented here.

Table 5.14. Pushover analysis results for Building 1B.

Step(i)	$u_{xN1}^{(i)} \text{ (m)}$	$V_{x1}^{(i)} \text{ (kN)}$	$d_1^{(i)} \text{ (m)}$	$a_1^{(i)} \text{ (m/s}^2\text{)}$
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.060323	1222.1	0.029498	0.821329
2	0.120323	2444.3	0.058838	1.642657
3	0.193448	3854.3	0.094596	2.590263
4	0.25729	4776.8	0.125814	3.210247
5	0.331839	5296.6	0.162268	3.559554
6	0.421031	5454.4	0.205883	3.665621
7	0.532104	5541.1	0.260198	3.723889
8	0.634588	5571.7	0.310312	3.74445
9	0.694588	5575.3	0.339652	3.746858
10	0.754588	5576.3	0.368992	3.747537
11	0.814588	5576.1	0.398332	3.747349
12	0.874588	5575.8	0.427671	3.747168
13	0.934588	5575.5	0.457011	3.746991
14	0.994588	5575.3	0.486351	3.746819
15	1.054588	5575.0	0.515691	3.746653
16	1.114588	5574.8	0.545031	3.746492
17	1.196528	5540.5	0.585099	3.723487
18	1.256949	5459.0	0.614645	3.668693
19	1.324935	5342.0	0.64789	3.590044
20	1.407412	5109.4	0.688221	3.433761
21	1.478228	4654.0	0.72285	3.127671
22	1.571281	4354.8	0.768353	2.926597
23	1.636607	4171.0	0.800297	2.803103
24	1.636607	4171.0	0.800297	2.803103

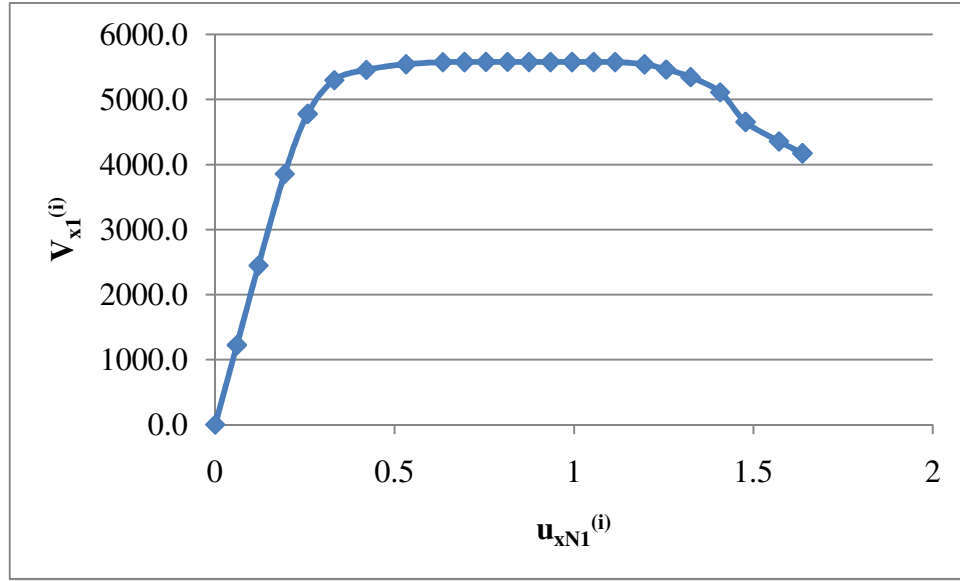


Figure 5.22. Pushover curve for Building 1B.

In the figure below, adapted pushover curve and modal spectrum curve can be observed.

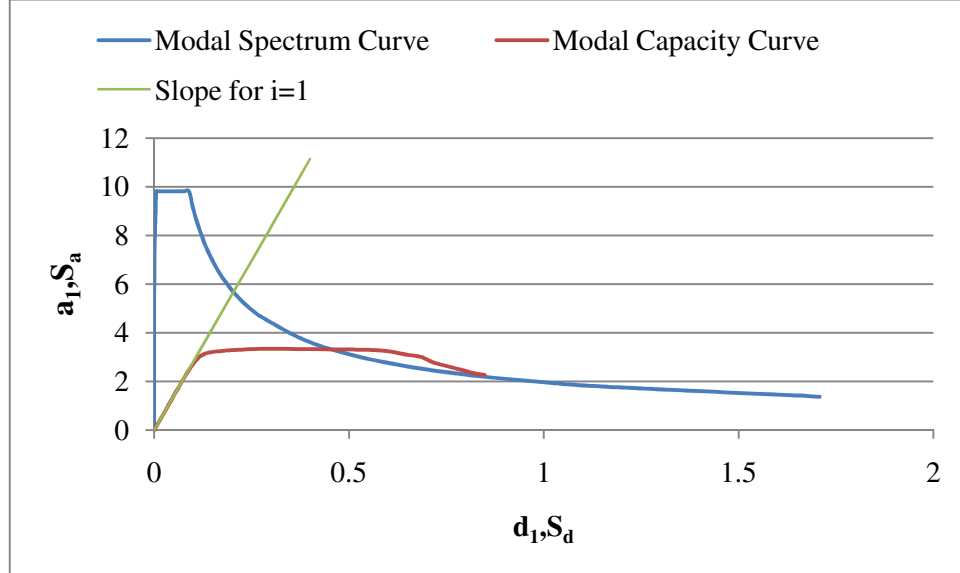


Figure 5.23. Modal spectrum and capacity curves for Building 1B.

Equation 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0526m * 38.853/m * 0.206m = 0.420m$$

The tip of the frame is ‘pushed’ to 0.420m in SAP2000. Plastic hinges formed in the frame at 0.420m displacement demand are shown in Figure 5.24.

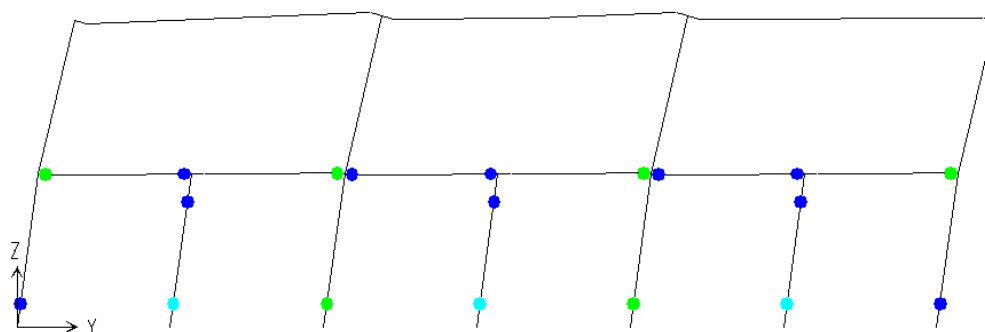


Figure 5.24. Plastic hinge formations for Building 1B at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.15. Plastic hinge curvature demands and performance levels for Building 1B.

Hinge	Θ_p	Φ_p	Φ_y	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0102	0.0204	0.0029	0.0233	0.0199	0.0023	0.0014	VDL
B101-R	-0.0034	-0.0068	-0.0029	-0.0097	0.0081	0.0014	0.0010	MDL
B102-R	-0.0104	-0.0208	-0.0029	-0.0237	0.0200	0.0026	0.0017	VDL
B103-L	0.0059	0.0118	0.0029	0.0147	0.0121	0.0017	0.0011	VDL
B103-R	-0.0029	-0.0048	-0.0029	-0.0077	0.0064	0.0012	0.0009	MDL
B104-R	-0.0104	-0.0208	-0.0029	-0.0237	0.0200	0.0026	0.0017	VDL
B105-L	0.0058	0.0116	0.0029	0.0145	0.0121	0.0017	0.0011	VDL
B105-R	-0.0030	-0.0060	-0.0029	-0.0089	0.0072	0.0013	0.0009	MDL
B106-R	-0.0157	-0.0314	-0.0029	-0.0343	0.0293	0.0037	0.0023	VDL
C101-B	0.0057	0.0114	0.0061	0.0175	0.0137	0.0030	0.0023	VDL
C102-B	0.0072	0.0144	0.0042	0.0186	0.0128	>0.004	0.0039	VDL
C102-T	-0.0036	-0.0072	-0.0042	0.0114	0.0082	0.0030	0.0025	MDL
C103-B	0.0082	0.0164	0.0051	0.0219	0.0163	>0.004	0.0039	VDL
C104-B	0.0072	0.0144	0.0042	0.0186	0.0128	>0.004	0.0039	VDL
C104-T	-0.0036	-0.0072	-0.0042	0.0114	0.0082	0.0030	0.0025	MDL
C105-B	0.0082	0.0164	0.0051	0.0219	0.0163	>0.004	0.0039	VDL
C106-B	0.0074	0.0148	0.0042	0.0190	0.0128	>0.004	0.0039	VDL
C106-T	-0.0036	-0.0072	-0.0042	0.0114	0.0082	0.0030	0.0025	MDL
C107-B	0.0044	0.0088	0.0048	0.0136	0.0099	0.0030	0.0025	MDL

5.7 Nonlinear Static Performance Analysis for Building 2B

In light of the results obtained from performance analysis of Buildings 1, 1A, 2 and 2A, an alternative design strategy is developed. According to this strategy, the short columns (C102,C104,C106) are provided with longitudinal reinforcement identical to top sections of middle columns (C103, C105). Therefore, short columns of Building 2B are provided with $28\Phi34+4\Phi32$.

Lateral load distribution is defined in SAP2000 and pushover analysis is performed. Pushover analysis results and pushover curve, along with the “adapted pushover curve” determined by using equations 2.27 and 2.28 of this study are shown below. Since modal analysis results are obviously same with Building 2, they are not presented here.

Table 5.16. Pushover analysis results for Building 2B.

Step(i)	$u_{xN1}^{(i)}$ (m)	$V_{x1}^{(i)}$ (kN)	$d_1^{(i)}$ (m)	$a_1^{(i)}$ (m/s ²)
			$=u_{N1}^{(i)}/\Phi_{xN1}\Gamma_{x1}$	$=V_{x1}^{(i)}/M_{x1}$
1	0.124432	4547.6	0.059909	3.097799
2	0.208753	6690.1	0.100507	4.557273
3	0.238579	6955.8	0.114867	4.738287
4	0.376477	7048.5	0.18126	4.801437
5	0.690645	6005.3	0.33252	4.090826

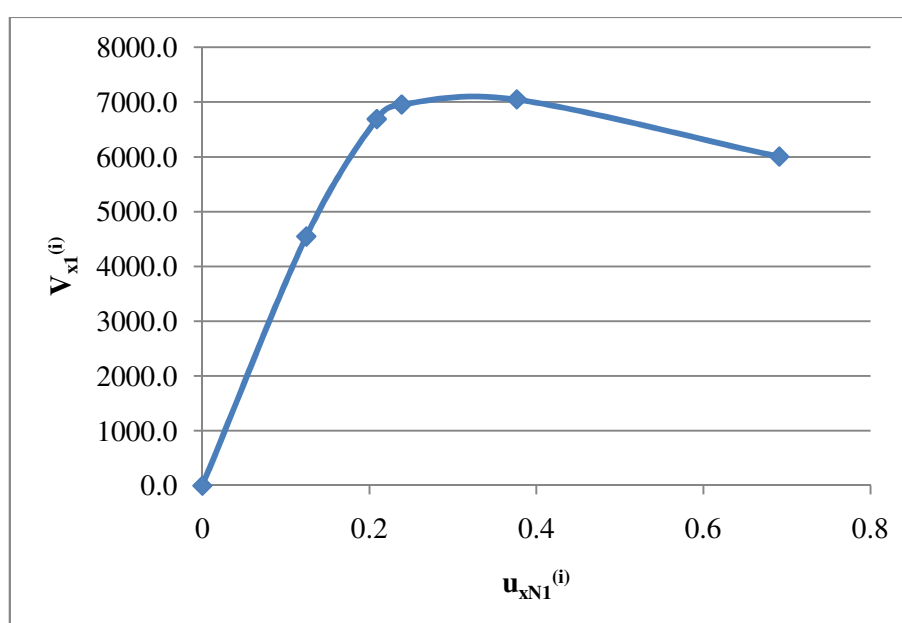


Figure 5.25. Pushover curve for Building 2B.

In the figure below, adapted pushover curve and modal spectrum curve can be observed.

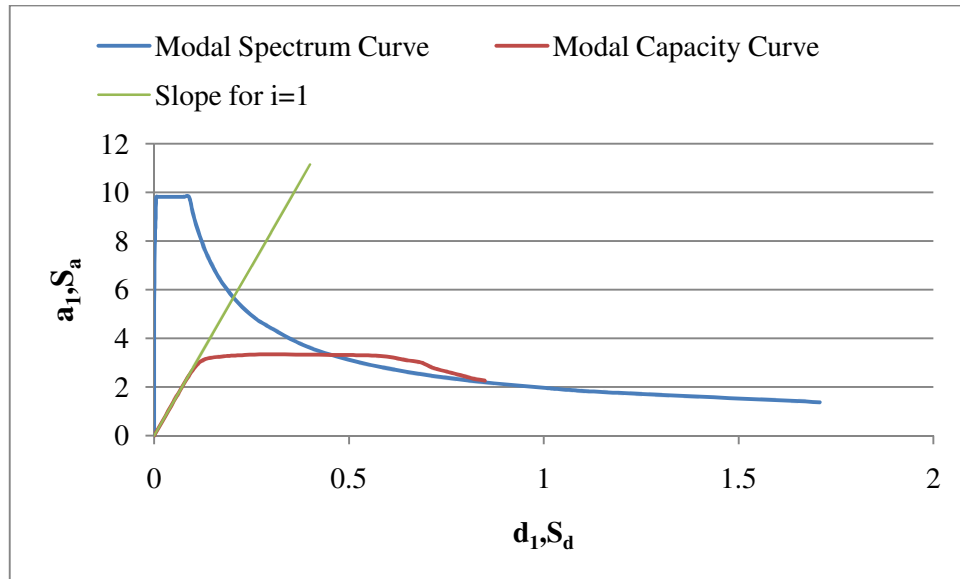


Figure 5.26. Modal spectrum and capacity curves for Building 2B.

Equation 2.30 and 2.31 yield;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} = 0.0539m * 38.543/m * 0.142m = 0.295m$$

The tip of the frame is ‘pushed’ to 0.295m in SAP2000. Plastic hinges formed in the frame at 0.295m displacement demand is shown in the figure below.

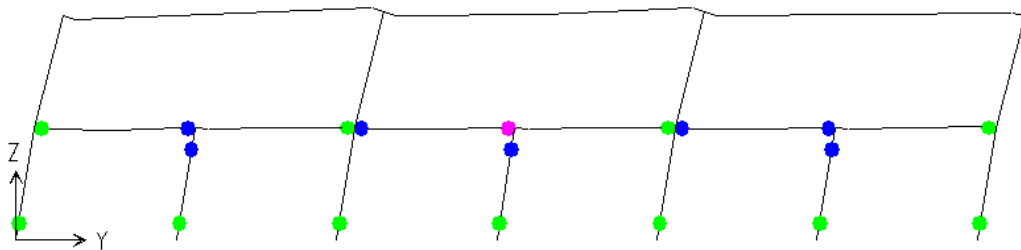


Figure 5.27. Plastic hinge formations for Building 2B at target displacement.

Hinge rotation results from SAP2000 are recorded. These results are then converted to curvatures using Equation 2.32 and total curvature demands are determined using Equation 2.33. These curvature values are then used to determine strain demands in confined and unconfined concrete and reinforcement steel.

Table 5.17. Plastic hinge curvature demands and performance levels for Building 2B.

Hinge	Θ_p	Φ_p	Φ_v	Φ_t	ϵ_s	ϵ_{cu}	ϵ_{cg}	D.L.
B101-L	0.0107	0.0214	0.0031	0.0245	0.0213	0.0022	0.0012	VDL
B101-R	-0.0017	-0.0034	-0.0028	-0.0062	0.0050	0.0011	0.0008	MDL
B102-R	-0.0079	-0.0158	-0.0028	-0.0186	0.0157	0.0022	0.0015	VDL
B103-L	0.0041	0.0082	0.0031	0.0103	0.0088	0.0013	0.0009	MDL
B103-R	-0.0015	-0.0030	-0.0028	-0.0058	0.0045	0.0010	0.0008	MDL
B104-R	-0.0079	-0.0158	-0.0028	-0.0186	0.0157	0.0022	0.0015	VDL
B105-L	0.0041	0.0082	0.0031	0.0103	0.0088	0.0013	0.0009	MDL
B105-R	-0.0015	-0.0030	-0.0028	-0.0058	0.0045	0.0010	0.0008	MDL
B106-R	-0.0147	-0.0294	-0.0028	-0.0322	0.0279	0.0033	0.0020	VDL
C101-B	0.0078	0.0184	0.0061	0.0245	0.0156	0.0040	0.0030	VDL
C102-B	0.0070	0.0165	0.0062	0.0227	0.0122	>0.004	0.0050	VDL
C102-T	-0.0042	-0.0099	-0.0062	-0.0161	0.0089	0.0040	0.0034	MDL
C103-B	0.0112	0.0263	0.0049	0.0312	0.0186	>0.004	0.0055	VDL
C104-B	0.0070	0.0165	0.0062	0.0227	0.0122	>0.004	0.0050	VDL
C104-T	-0.0042	-0.0099	-0.0062	-0.0161	0.0089	0.0040	0.0034	MDL
C105-B	0.0112	0.0263	0.0049	0.0312	0.0186	>0.004	0.0055	VDL
C106-B	0.0070	0.0165	0.0062	0.0227	0.0122	>0.004	0.0050	VDL
C106-T	-0.0042	-0.0099	-0.0062	-0.0161	0.0089	0.0040	0.0034	MDL
C107-B	0.0067	0.0158	0.0051	0.0209	0.0125	0.0040	0.0032	VDL

6. RESULTS

In this chapter, results obtained from linear and nonlinear performance analyses of the structures are summarized and compared in terms of various parameters. Performance levels of the structures are also evaluated.

6.1 Summary of Results

The results obtained by linear and nonlinear performance analyses of the structures, as explained in Chapters 4 and 5, are tabulated below.

Table 6.1. Performance levels of beams with linear and nonlinear methods.

Beam		B101	B102	B103	B104	B105	B106
Building 1	Linear	MDL	MDL	MDL	MDL	MDL	MDL
	Nonlin	VDL	VDL	VDL	VDL	VDL	VDL
Building 1A	Linear	VDL	VDL	VDL	VDL	VDL	VDL
	Nonlin	VDL	VDL	VDL	VDL	VDL	VDL
Building 1B	Nonlin	VDL	VDL	VDL	VDL	VDL	VDL
Building 2	Linear	MDL	MDL	MDL	MDL	MDL	MDL
	Nonlin	VDL	VDL	MDL	VDL	MDL	VDL
Building 2A	Linear	VDL	VDL	MDL	VDL	MDL	VDL
	Nonlin	VDL	VDL	VDL	VDL	VDL	VDL
Building 2B	Nonlin	VDL	VDL	MDL	VDL	MDL	VDL

Table 6.2. Performance levels of column ends with linear and nonlinear methods.

[illegible]

Table 6.3. Performance levels of columns with linear and nonlinear methods neglecting relative storey drifts.

Column	Building 1		Building 1A		Buil 1B	Building 2		Building 2A		Buil 2B
	Lin	Nonl	Lin	Nonl	Nonl	Lin	Nonl	Lin	Nonl	Nonl
C101	MDL	VDL	MDL	MDL	VDL	VDL	VDL	MDL	VDL	VDL
C102	MDL	VDL	MDL	VDL	VDL	VDL	VDL	VDL	VDL	VDL
C103	VDL	VDL	MDL	MDL	VDL	VDL	VDL	MDL	VDL	VDL
C104	MDL	VDL	MDL	VDL	VDL	VDL	VDL	VDL	VDL	VDL
C105	VDL	VDL	MDL	MDL	VDL	VDL	VDL	MDL	VDL	VDL
C106	MDL	VDL	MDL	VDL	VDL	VDL	VDL	VDL	VDL	VDL
C107	MDL	VDL	MDL	MDL	MDL	MDL	VDL	MDL	VDL	VDL
C201	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL
C202	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL
C203	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL
C204	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL	MDL

Table 6.4. Relative storey drifts and performance levels of buildings with linear elastic methods.

Column	δ_{ij}/h_{ij}							
	Bu1	P.L.	Bu1A	P.L.	Bu2	P.L.	Bu2A	P.L.
C101	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C102	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C103	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C104	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C105	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C106	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C107	0.017	VDL	0.008	MDL	0.018	VDL	0.008	MDL
C201	0.031	SDL	0.018	VDL	0.030	SDL	0.017	VDL
C202	0.031	SDL	0.018	VDL	0.030	SDL	0.017	VDL
C203	0.031	SDL	0.018	VDL	0.030	SDL	0.017	VDL
C204	0.031	SDL	0.018	VDL	0.030	SDL	0.017	VDL

6.2 Performance Evaluation of Structures

Results obtained from linear and nonlinear analyses and tabulated in previous section are used to evaluate overall performance levels of structures as per TSC2007 Section 7.7.

- Building 1

- (i) Linear Elastic Method

All beams of Building 1 are in *Minimum Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Significant Damage Level*. According to TSC2007 Section 7.7.3.(b), which limits shear force resisted by columns that are in Significant Damage Level, the building is in *Collapse Prevention Performance Level*.

- (ii) Nonlinear Static Method

All beams of Building 1 are in *Visible Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. According to TSC2007 Section 7.7.3.(c) and 7.7.4.(d), which limits shear force resisted by columns whose both joints exceed Minimum Damage Level, the building is in *Collapse Level*.

- Building 1A

- (i) Linear Elastic Method

All beams of Building 1A are in *Visible Damage Level*, where all first storey columns are in *Minimum Damage Level* and all second storey columns are in *Visible Damage Level*. The building satisfies TSC2007 Section 7.7.3 and hence it is in *Life Safety Performance Level*.

- (ii) Nonlinear Static Method

All beams of Building 1A are in *Visible Damage Level*, where 3 out of 7 (43%) first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. Since only one joint of the columns are in Visible

Damage Level, the building satisfies TSC2007 Section 7.7.3.(c) and hence it is in *Life Safety Performance Level*.

- Building 1B

- (i) Nonlinear Static Method

All beams of Building 1B are in *Visible Damage Level*, where 6 out of 7 (86%) first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. Since only one column has both joints in Visible Damage Level and the building satisfies TSC2007 Section 7.7.3.(c), it is in *Life Safety Performance Level*.

- Building 2

- (i) Linear Elastic Method

All beams of Building 2 are in *Minimum Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Significant Damage Level*. According to TSC2007 Section 7.7.3.(b), which limits shear force resisted by columns that are in Significant Damage Level, the building is in *Collapse Prevention Performance Level*.

- (ii) Nonlinear Static Method

4 out of 6 (67%) beams of Building 2 are in *Visible Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. According to TSC2007 Section 7.7.3.(c) and 7.7.4.(d), which limits shear force resisted by columns whose both joints exceed Minimum Damage Level, the building is in *Collapse Level*.

- Building 2A

- (i) Linear Elastic Method

4 out of 6 (67%) beams of Building 2A are in *Visible Damage Level*, where 3 out of 7 (43%) first storey columns are in *Visible Damage Level* and all second storey

columns are in *Visible Damage Level*. Since only one joint of the columns are in Visible Damage Level, the building satisfies TSC2007 Section 7.7.3.(c) and hence it is in *Life Safety Performance Level*.

(ii) Nonlinear Static Method

All beams of Building 2A are in *Visible Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. Since only one joint of the columns are in Visible Damage Level, the building satisfies TSC2007 Section 7.7.3.(c) and hence it is in *Life Safety Performance Level*.

- Building 2B

(i) Nonlinear Static Method

4 out of 6 (67%) beams of Building 2B are in *Visible Damage Level*, where all first storey columns are in *Visible Damage Level* and all second storey columns are in *Minimum Damage Level*. Since only one joint of the columns are in Visible Damage Level, the building satisfies TSC2007 Section 7.7.3.(c) and hence it is in *Life Safety Performance Level*.

According to TSC2007 Table 7.7, target performance level for the type of buildings analyzed in this study (industrial buildings) subject to an earthquake that has 10% probability to occur in 50 years, is *Life Safety Performance Level*. In Table 6.5 below, overall performance levels of the buildings analyzed are tabulated and checked if they satisfy target performance level.

Table 6.5. Overall performance levels of buildings.

Building	Performance Level			
	Linear Elastic Method	Check	Nonlinear Static Method	Check
Building 1	Collapse Prevention	X	Collapse	X
Building 1A	Life Safety	✓	Life Safety	✓
Building 1B	-	-	Life Safety	✓
Building 2	Collapse Prevention	X	Collapse	X
Building 2A	Life Safety	✓	Life Safety	✓
Building 2B	-	-	Life Safety	✓

7. CONCLUSIONS AND RECOMMENDATIONS

Results obtained from linear and nonlinear performance analyses, and summarized in Chapter 6, are interpreted to draw the following conclusions:

(i) Relative storey drift limits govern seismic performance for this type of pre-cast buildings. Buildings analyzed and designed using uncracked section rigidities do not satisfy target performance (Life Safety) level. On the other hand, buildings analyzed and designed using cracked section rigidities satisfy target performance level.

TSC2007 Section 3.2.3 states that “in structural analyses in which methods described in Chapter 2 are used, *uncracked section rigidities* shall be considered”. On the other hand, TSC2007 Section 7.4.13 states that “for members subject to bending, *effective (cracked) section rigidities* shall be used”.

Due to special aspects of pre-cast concrete structures, more specifically big spans and storey heights and columns hinged at the top as described in Chapter 1, column dimensions are usually governed by relative storey drift limits. Buildings 1 and 2, which are designed considering uncracked section rigidities, do not satisfy relative storey drift limits for the target performance although they satisfy. However, Buildings 1A and 2A, which are designed considering cracked section rigidities, satisfy relative storey drift limits for target performance. Results obtained from analyses of these four frames reveal that in order to satisfy target performance (Life Safety) level for these types of pre-cast structures, initial analysis and design should be performed considering cracked section rigidities.

(ii) Nonlinear static (pushover) analysis method is not reliable for this type of structures because of the difference of Structural Behaviour Factor (R) values at storey levels.

The type of pre-cast concrete structures investigated in this study has differing Structural Behaviour Factors at storey level and roof level, as described in detail in Section 1.2.(ii). This is because the columns are connected to beams with moment-resisting connections at storey level and with hinged connections at roof level.

Structural Behaviour Factor, R , defined in TSC2007 is the reduction factor that accounts for nonlinear behaviour of the structural system. In other words, seismic forces, obtained using *elastic* response spectra, are divided by R values, before they are applied to the system to take into account nonlinear behaviour. In nonlinear static analysis method, however, the ductility demand is determined by equal displacement rule, considering solely the first mode of the structure. R values specified in TSC2007 can be described as ‘anticipated ductility demands’ and a structure designed using the procedure given in TSC2007 for differing R values along its height, obviously cannot be represented realistically with such a ‘simplified’ method. Nonlinear dynamic analysis methods, such as *Incremental Modal Pushover Analysis* and *Time History Analysis* would yield more reliable results. Therefore, it is highly recommended that the results obtained in this study are supported utilizing one of the methods mentioned above.

(iii) Strengthening one-storey columns may improve seismic performance of this type of pre-cast structures in non-linear methods. This is due to the fact that columns that do not extend to the next floor do not need to satisfy strong column-weak beam criterion.

Very large spans and loads yield very strong beams and hence very strong column sections at beam levels to satisfy “strong column-weak beam” criterion. One-storey columns do not need to satisfy “strong column-weak beam” criterion and therefore have significantly smaller ultimate strength. This causes an “accumulation” of plastic rotations at one-storey column plastic hinge regions and thus excessive curvature demands, which consequently causes both joints of one-storey columns to exceed Minimum Damage Level. When both joints of one-storey columns exceed minimum damage level, the structure does not satisfy shear force ratio limit for such columns and thus the building does not satisfy target performance level.

To overcome this inadequacy, a design strategy is developed. According to this strategy, required longitudinal reinforcement at middle columns that extend to the roof, namely C103 and C105, to satisfy strong column-weak beam criterion, is provided at bottom and top sections of one-storey columns.

The structures in which this design strategy is applied are Building 1B and Building 2B. It can be seen from Table 6.5 that Buildings 1B and 2B satisfy target performance in non-linear performance analysis method. This is due to the fact that in Buildings 1B and 2B, only one joint of one-storey columns exceed Minimum Damage Level (see Table 6.2) and thus these buildings satisfy the condition that limits shear force ratio on columns with both plastic regions exceeding Minimum Damage Level. However, it is obvious that further research is needed to support this hypothesis and perhaps develop mathematical expressions to propose longitudinal reinforcement requirements for this type of pre-cast structures.

(iv) Beam sections perform well

It should be noted that no beam section in any analysis method or for any structure exceeds Visible Damage Level. This is due to the fact that design of beam members is governed by gravitational loading analysis, considering very large spans and loadings. At initial design stage, the required longitudinal reinforcement is determined by using combinations including $1.4G+1.6Q$ and also by TSC2007 Section 3.12.2.2 specification that “welded connections of pre-cast members shall be designed considering seismic forces, determined by Chapter 2, multiplied by two”. These two conditions of initial design cause the beams to perform well in seismic performance evaluation, considering gravitational loads taken into account in seismic performance analyses are determined in accordance with masses calculated for seismic analysis, meaning gravitational loads are reduced in seismic performance analysis methods.

(v) Nonlinear analysis method generally yields more conservative results compared with linear analysis method

Results tabulated in Table 6.1 and Table 6.2 reveal that nonlinear analysis method yields more conservative results for 50% of the beam sections and 23.6% of the column sections.

(vi) Ductility of members and beam-column connections are adequate. In other words, no shear failure occurs for any of the buildings.

Since the buildings are analyzed and designed in accordance with TSC2007, it is no surprise that shear failure and thus brittle behaviour does not occur in any of the buildings.

(vii) In nonlinear static performance analysis method, reinforcing steel strain governs seismic performance of members.

Plastic rotations at target displacement and thus plastic rotation and curvature demands are tabulated in Table 5.3, Table 5.6, Table 5.9, Table 5.11, Table 5.13, Table 5.15 and Table 5.17. Performance levels of sections are governed by reinforcing steel strain exceeding specified limits for all cases.

In light of the results obtained, further recommendations can be made. These recommendations are listed below.

(i) In this study, 2D frame models are used to focus on seismic performance of sections. Therefore, lack of diaphragm behaviour at the roof and the effect of 30% of the seismic forces at orthogonal direction are neglected. To represent more realistic behaviour of the structures, 3D mathematical models for this type of structures should be used.

(ii) In this study, four frames for linear elastic method are six frames for nonlinear static method are used. Analyzing numerous frames with differing dimensions and design parameters would obviously provide more trustworthy conclusions.

(iii) In practice, pre-cast concrete design is made by typifying “design regions” such as connections and members, choosing representative design regions and applying the proposed design for all members and connections of the same type. This fact partially makes nonlinear static analysis method easier relative to other structure types, since the engineer would need to analyze less section types. Supported by the fact that nonlinear static analysis method yields more conservative results, it is recommended in this study that nonlinear static analysis method is used for performance evaluation of this type of pre-cast structures.

(iv) Although it is stated previously that further research is needed to strengthen this study's conclusions, it is highly recommended that this type of pre-cast structures are analyzed and designed considering cracked section stiffnesses. However, using cracked section stiffness for members yields excessively large column sections. (60% larger for Building 1 and 53% larger in Building 2) In pre-cast construction, section dimensions are very important, because larger section dimensions increase transportation and erection costs. Therefore, it is also recommended that other design strategies, rather than using cracked section stiffnesses, are developed for this type of pre-cast structures.

(v) Although TSC2007 states that "columns that do not extend to the next floor do not need to satisfy strong column-weak beam criterion", it is highly recommended that such columns of this type of buildings satisfy the criterion to improve seismic performance.

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