The Advantage of High-Strength Concrete (HSC) in High-Rise Buildings:

From Area Saving Aspect

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A Thesis Submitted to

The School of Civil and Environmental Engineering

Presented in Partial Fulfillment of the Requirements for the Degree of

Master of Science (Structural Engineering)

Addis Ababa University

Addis Ababa, Ethiopia

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Approved by Board of Examiners

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Chairperson	Signature	Date

ABSTRACT

The Advantage of High-Strength Concrete (HSC) in High-Rise Buildings: From Area Saving Aspect

Sinit Tsegaye

Addis Ababa University, 2014

In recent years, the shortage and value of land in terms of cost and asset has increased the need to use land effectively and efficiently. This is more significant in High-Rise multi story buildings where small unusable dimension from each floor will accumulate and add up to huge loss of usable area.

Based on this study, it has been concluded that, the area saved while using HSC to NSC columns and shear walls is more significant as the height of the building increases. The % decrease in concrete is higher, % decrease in reinforcement is lower and the total % decrease in cost is lower. Thus the higher the building, the amount of area and concrete saved is significant thus allowing the user to have more usable floor area leading to an economical use of land.

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HSC – High-Strength Concrete

NSC – Normal-Strength Concrete

ULS – Ultimate Limit State

SLS – Serviceability Limit State

ETABS – Extended 3D Analysis of Building Systems Software

DL – Dead Load

LL – Live Load

EQ – Earth Quake (Static Seismic Load)

WL – Wind Load

DCL- Ductility Class Low

DCM – Ductility Class Medium

DCH – Ductility Class High

RH – Relative Humidity

LIST OF NOTATIONS

Latin upper case letters

A_c – Cross-sectional area of concrete

A – Cross-sectional area of the shape for second moment of area

C_1 – Factor having different values for different structures

C_{pe} - External pressure coefficient

$C_e (Z)$ – Exposure coefficient

$C_t (Z)$ – Topographic coefficient

$C_r (Z)$ - Roughness coefficient

C_x and C_y – Factors for adjusting span moments

D – Total slab or beam depth

E_{cm} – Modulus of elasticity of concrete

E_c – Tangent modulus

E_{cd} – Design value of the modulus of elasticity of concrete cantilever, twice the length to the face of the support

F_b – total seismic base shear force of building for each main direction

F_t – Concentrated shear force at the top of the building

H – Height of the building above the base

I – Importance factor

I_c – Second moment of area (uncracked concrete section) of bracing members

I_x – Second moment of area with respect to the x-axis

I_y – Second moment of area with respect to the y-axis

$I_{x'}$ – Second moment of area with respect to its centroidal axis x'

$I_{y'}$ – Second moment of area with respect to its centroidal axis y'

K_T – Terrain factor

L – Total height of building above level of moment restraint

L_e – Shorter span length for two way slab

L_o – Distance in meter between points of zero moments, and for a

L_y – Longer span of a panel

L_x – Shorter span of a panel

L_u - Actual length of an upwind slope

P_d – Total design load

P_{tot} – The total gravity load at and above the story considered in the seismic design situation

S – Site coefficient

$S_d(T)$ – Ordinate of the design spectrum

T_1 – Fundamental Period of vibration

V_{tot} – Total seismic story shear

V_s – Shear force on beams

V_{ref} – Reference wind velocity

W – Total vertical load of building

W_e – External wind pressure

Z_e – Reference height

Z_o – Roughness length

Z_{min} – Minimum height

Latin lower case letters

b – Building breadth

d – Effective slab or beam depth

d_r – Design inter-story drift, evaluated as the difference of the average lateral displacements

d_y – Perpendicular distance between the x' and x axes

d_x – Perpendicular distance between the y' and y axes

f_{ck} – Characteristic compressive strength of concrete

f_{cm} – Mean value of concrete compressive strength

f_{cd} – Design compressive strength

f_{ctd} – Design tensile strength of concrete

f_{ctm} – Mean value of axial tensile strength of concrete

f_{ctk} – Characteristic tensile strength of concrete

f_{yk} – Characteristic yield strength of reinforcement bars

h – Inter story height

k_1 – Relative flexibilities of rotational restraints

k_D - factor for ductility class

k_r - factor for regularity in elevation

k_w - factor reflecting the prevailing failure mode in structural system with walls

m_i – Design moment per unit width at the point of reference

n_s – Number of stories

q_{ref} - Reference wind pressure

u – Perimeter of a given area

Greek lower case letters

$\phi(t,t_0)$ – Creep coefficient

ϵ_{cs} - Basic shrinkage strain

$\phi(\infty,t_0)$ – Final creep coefficient

σ_c – Stress in concrete

ε_c – Strain in concrete at a given stress

ε_{c1} – Strain of concrete at peak stress

η – Ratio between the strain in concrete at a given stress and strain in concrete at peak stress

γ_C – Partial safety factor for concrete

a_{cc} – Coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied

a_{ct} - Coefficient taking account of long term effects on the tensile strength and of unfavorable effects resulting from the way the load is applied

ε_{c2} – Strain reaching the maximum strength for Parabola-Rectangle stress-strain relation

ε_{cu2} – Ultimate strain for Parabola-Rectangle stress- strain relation

ε_{c3} – Strain reaching the maximum strength for Bi-linear stress-strain relation

ε_{cu3} – Ultimate strain for Bi-linear stress-strain relation

λ – Effective height of the compression zone of concrete under compressive force

σ_{all} – Allowable bearing capacity of soil

θ – Inter-story drift sensitivity coefficient

γ - Behavior factor

v - is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement

a – Design bedrock acceleration of gravity

a_o – Bedrock acceleration ratio for the site depending on the seismic zone

β – Design response factor

γ_o - basic value of the behavior factor which depends upon the structural type

β_a – Appropriate constant term for slabs and beams

a_i - Bending moment coefficients for rectangular panels supported on four sides (a_{xs}, a_{xf}, a_{ys} and a_{yf} where s = support, f = field, x = direction of shorter span y = direction of longer span)

β_v – Shear force coefficient.

ρ – Air density

Φ - Upwind slope

CHAPTER 1—GENERAL

1.1—INTRODUCTION

The definition of High-Strength Concrete (HSC) is subjective and varies on a geographical basis. In regions where concrete with a compressive strength of 62 MPa is already being produced commercially, High-Strength Concrete (HSC) might be in the range of 83 to 103 MPa. However, in regions where the upper limit on commercially available material is currently 34 MPa concrete, 62 MPa concrete is considered as High-Strength. Material selection, concrete mix proportioning, batching, mixing, transporting, placing, and curing procedures are applicable across a wide range of concrete strengths. Different literatures define High-Strength Concrete (HSC) as follows:-

- "A High-Strength Concrete (HSC) is a concrete having a uniaxial compressive strength in the range of about 55 MPa to 138 MPa or higher." Arthur H.Nilson
- "A High-Strength Concrete (HSC) is concrete which has a compressive strength greater than 50 MPa." RF. Warner
- "Concrete with 28 day strengths in excess of 6000psi are referred to as High-Strength Concretes (HSC)." Mc.Gregor
- "A High-Strength Concrete (HSC) is a concrete which has a compressive strength for design of 41 MPa or greater." ACI363R-92

In Ethiopia, according to EBCS-2, the highest compressive strength of concrete is given as 60 MPa.

1.2—SCOPE

This thesis tries to investigate the advantage of using High-Strength Concrete (HSC) columns and shear-walls in buildings as compared with buildings with Normal-Strength Concrete (NSC) so as to increase net floor area. This advantage of HSC is found in different literatures in relation to various building elements.

"In recent years, there has been a significant increase in the utilization of HSC with compressive strength in range of 50 to 100 MPa in columns and core walls of buildings. Use of HSC reduces the column construction coast and increases the net floor space available for use. ACI Committee 368 (1992) has indicated that using HSC with a minimum of longitudinal steel (i.e. 1 percent) is the most economical solution." (RF Warner, 1999)

"The most common application of HSC has been in the columns of tall concrete buildings, where normal concrete would result in unacceptably large cross sections, with loss of valuable floor space. It has been shown that the use of the more expensive HSC mixes in columns not only saves floor are but also is economical than increasing the amount of steel reinforcement." (Nilson, 2010)

"The expensive use of reinforced concrete construction, especially in developing countries, is due to its relatively low cost compared to other materials such as structural steel. The cost of construction changes with the region and strongly depends on the local practice. Even if it seems cheap, the cost of space that we sacrifice for the structural elements in high-rise buildings becomes very high, especially if long term benefit is evaluated.

If a specific objective criterion can be expressed mathematically, then optimization techniques may be employed to obtain a maximum or minimum for the objective function. Optimization procedures and techniques comprise an entire subject such as weight and cost. The criterion of minimum weight is emphasized throughout, under the general assumption that minimum material represents minimum cost. Other subjective criteria must be kept in mind, even though the integration of behavioral principles with design of structural steel elements utilized simple objective criteria, such as weight or space saving." (Salmon, 2004).

"Currently it was able to develop concrete with high compressive strength, up to 40 MPa and higher, which greatly helps in finding small structural element dimensions from design. Especially to build high-rise buildings by reducing column size and increasing available space, HSC plays a significant role (NRMCA 2001). In recent years, the application of HSC has increased and HSC has been possible as a result recent development in material technology and a demand for it, in relation to its space saving." (ACI 363R-92).

1.3—LITRATURE REVIEW

1.3.1—Historical background of High-Strength Concrete (HSC)

The use and definition of High-Strength Concrete (HSC) has seen a gradual and continuous development over many years. Concrete with a compressive strength of 34 MPa was considered as High-Strength in 1950s. Whereas, in 1960s, concrete used in reinforced concrete construction had strengths in the range of 41 and 52 MPa. In the early 1970s, 62 MPa concrete was produced. Today, compressive strengths approaching 138 MPa have been used in castin-place buildings. As material technology and production processes evolve, it is likely that the maximum compressive strength of concrete will continue to increase and HSC will be used widely in different applications.

Demand for and use of HSC for tall buildings began in the 1970s, primarily in the U.S.A. Water Tower Place in Chicago, IL, which was completed in 1976 with a height of 260 m and having columns and shear walls with compressive strength of 62 MPa. The 311 South Wacker building in Chicago, completed in 1990 with a height of 293 m, used 83 MPa specified compressive strength concrete for the columns. In their time, both buildings held the record for the world’s tallest concrete building. Two Union Square buildings in Seattle, WA, completed in 1989, hold the record for the highest specified compressive strength concrete used in a building at 131 MPa.

HSC is widely available throughout the world, and its use continues to spread, particularly in the Far and Middle East. All of the tallest buildings constructed have some structural contribution from HSC in vertical column and wall elements. Petronas Towers 1 and 2 completed in 1998 in Kuala Lumpur, Malaysia, used concrete with specified cube strengths up to 80 MPa in columns and shear walls both at 452 m. Taipei 101 in Taiwan, which was completed in 2004 has a total height of 509m. This structural system uses a mix of steel and concrete elements, with specified concrete compressive strengths up to 69 MPa in composite columns. All these height records have been suppressed by a taller building. The present record is held by the tallest building and the tallest structure of any type in the world, the Burj Khalifa in Dubai, United Arab Emirates, which has a total height of 828m.

1.3.2—Properties of High-Strength Concrete (HSC)

1.3.2.1—Selection of Materials

The production of HSC should meet the requirements of workability and strength than Lower-Strength Concretes. HSC can be made using a wide range of carefully selected but widely available materials. These materials include, cement, aggregates, water and chemical or mineral admixtures.

The choice of *Cement* for High-Strength Concrete (HSC) is extremely important. Furthermore, within a given cement type, different products will have different strength development characteristics because of the variations in compound composition and fineness. Strength development will depend on both cement characteristics and cement content. The use of high cement content will ordinarily produce higher ultimate strength. In general, however, no more cement should be used than necessary for required workability which is defined as the property of freshly mixed concrete which determines the ease and homogeneity with which it can be mixed, placed, compacted, and finished.

Both *Fine and Coarse Aggregates* may be used in the production of HSC. According to ACI Committee Report 363R-92, fine aggregates with a rounded particle shape and smooth texture have been found to require less mixing water in concrete and for this reason are preferable in HSC. The optimum gradation of fine aggregate for HSC is determined more by its effect on water requirement than on physical packing.

The requirements for *Water* quality for HSC are the same as those for conventional concrete. Usually, water for concrete is specified to be of potable quality. This is certainly conservative but usually does not constitute a problem since most concrete is produced near a municipal water supply.

In addition to higher strength in compression, most other engineering properties of HSC are improved by the use of *Chemical or Mineral Admixtures*. Selection of type, brand, and dosage rate of all admixtures should be based on performance with the other materials being considered or selected for use on the project. Significant increases in compressive strength, control of rate of hardening, accelerated strength gain, improved workability, and durability are contributions that can be expected from the admixtures chosen.

1.3.2.2—Concrete mix proportions

Concrete mix proportions for HSC have varied widely depending upon many factors. The strength level required, test age, material characteristics, and type of application have influenced mix proportions. In addition, economic considerations, structural requirements, manufacturing practicality, anticipated curing environment, and even the time of year do affect the selection of mix proportions.

The selection of mix proportions can be influenced by the *Testing Age*. This testing age has varied depending upon the construction requirements. Most often the testing age has been thought to be the age at which the acceptance criteria is established, for example at 28 days. According to ACI 363R-92 Committee Report, testing is conducted prior to the age of acceptance testing or after that age depending upon the type of information required.

Early Age: - Pre-stressed concrete operations may require high strengths in 12 to 24 hours. Special applications for early use of machinery foundations, pavement traffic lanes, or slip formed concrete have required high strengths at early ages. Post-tensioned concrete is often stressed at ages of approximately 3 days and requires relatively high strengths. The optimum materials selected, and therefore the mix proportions, may vary for different test ages. Early-age strengths may be more variable due to the influence of curing temperature and the early-age characteristics of the specific cement. Therefore, anticipated mix proportions should be evaluated for a higher required average strength or a later test age.

Twenty-eight days: - A very common test age for compressive strength of concrete has been 28 days. Performance of structures has been empirically correlated with moist-cured concrete cylinders, usually 6 x 12 in. (152 x 305 mm) prepared according to ASTM C 31 and C 192. This has produced good results for concretes within lower strength ranges not requiring early strengths or early evaluation. High-strength concretes gain considerable strengths at later ages and, therefore, are evaluated at later ages when construction requirements allow the concrete more time to develop strengths before loads are imposed. Proportions, notably cementitious components, have usually been adjusted depending upon test age.

Later age: - High-strength concretes are frequently tested at later ages such as 56 or 90 days. High-strength concrete has been placed frequently in columns of high-rise buildings. Therefore, it has been desirable to take advantage of long-term strength gains so that efficient use of construction materials can be achieved. This has often been justified in high-rise buildings where full loadings may not occur until later ages.

The relationship between *water-cement ratio* and compressive strength, which has been identified in conventional concrete, has been found to be valid for HSC. Higher cement and lower water contents have produced higher strengths. Proportioning larger amounts of cement into the concrete mixture, however, has also increased the water demand of the mixture.

In proportioning a concrete mixture, it is generally agreed that the *fine aggregates* have considerably shown more impact on mix proportions than the coarse aggregates. The fine aggregates contain a much higher surface area for a given weight than do the larger coarse aggregates. Since the surface area of all the aggregate particles must be coated with a cementitious paste, the proportion of fine to coarse can have a direct quantitative effect on paste requirements. The optimum amount and size of coarse aggregate for given sand will depend to a great extent on the characteristics of the sand.

1.3.2.3 — Batching, mixing, transporting, placing and curing

The batching, mixing, transporting, placing, and curing procedures for HSC are not different in principle from those procedures used for conventional concrete. But, some changes, some refinements, and some emphasis on critical points are necessary. Maintaining the water content as low as possible and consistent with placing requirements, is good practice for all concrete but is critical for HSC. The production of HSC normally involves the use of relatively large unit cement content. In addition, the production and testing of HSC requires well-qualified concrete producers and testing laboratories.

The age at which concrete must develop its high-strength is an important factor in the design of a mix. If a high ultimate strength i.e. strength at 28 days or later is the aim, the steps available to achieve it include the use of high cement content, low water to cement ratio and the use of water-reducing admixtures.

HSC may be mixed entirely at the batching plant, in a central or truck mixer or by a combination of the two and transported by a variety of methods and equipment such as truck mixers, stationary truck bodies with and without agitators, pipeline or hose, or conveyor belts. Each type of transportation has specific advantages and disadvantages depending on the conditions of use, mixture ingredients, accessibility and location of placing site, required capacity and time for delivery, and weather conditions.

Preparations for placing HSC should include recognition at the start of the work that certain abnormal conditions might exist. These conditions will require some items of preparation that cannot be provided readily the last minute before concrete is placed. Since workability time is expected to be reduced, preparation must be made to transport, place, consolidate, and finish the concrete at the fastest possible rate. In other words, delivery of concrete to the job site must be scheduled so it will be placed promptly on arrival.

Equipment for placing the concrete must have adequate capacity to perform its functions efficiently so there will be no delays at distance portions of the work. There should be ample vibration equipment and manpower to consolidate the concrete quickly after placement in difficult areas. All equipment should be in the first class operating condition so as to avoid Breakdowns or delays that stop or slow the placement which can seriously affect the quality of the work. Accordingly, provision should be made for an ample number of standby vibrators, at least one standby for each three vibrators in use.

After the placing of concrete, the curing process which is critical in the production of quality HSC continues. Curing is the procedure of maintaining satisfactory moisture content and a favorable temperature in concrete during the hydration period of the cementitious materials so that the desired properties of the concrete can be developed. The potential strength and durability of concrete will be fully developed only if it is properly cured at an early age for an adequate period prior to being placed in service.

1.3.2.4—Quality control

The cost of HSC is mainly influenced by the selection of materials, increased testing, quality control, and inspection. Since quality and consistency of the concrete is crucial, additional steps must be taken to insure that quality and consistency are met.

The contractor needs to have a quality control person at the site to control both the scheduling of trucks and the consistency of the concrete at the time it was delivered. No water should be added to the trucks after they had come onto the site and that any minor adjustments would be made prior to sending the truck to the site. Furthermore, a full-time technician should be employed to carry out sampling and testing on the site. This quality control ultimately creates better concretes of all strengths and result in better and more economical use of materials.

Generally, the economic advantages of HSC are most readily realized when the concrete is used in the columns and shear walls of high-rise buildings. In this application, engineers may take full advantage of its increased compressive strength, reducing the amount of reinforcement steel, reducing column and wall sizes to increase usable floor space or allowing additional stories. These benefits overshadow the increased quality control costs and possible higher cost of raw materials.

1.3.3—Mechanical Properties of Normal and High- Strength Concrete

- **Grades of Concrete**

Concrete generally is graded in terms of its characteristic compressive cube strength. The grade of concrete to be used in design depends on the classification of the concrete work and intended use. The numbers in the grade designation denote the specified characteristic compressive strength in MPa. For example, C30 or C95 represent concrete which has a compressive strength of 30 MPa and 95 MPa respectively.

- **Characteristic Compressive Strength**

The characteristic compressive strength of concrete is denoted by that strength (*cylindrical* f_{ck} or *cubic* f_{ck}) below which 1 out of 20 or 5% of all possible strength measurements may be expected to fall. According to EBCS-2, the compressive strength of concrete is determined from tests on 150 mm cubes at the age of 28 days. Cylindrical or cubical specimens of other sizes may also be used with conversion factors determined from a comprehensive series of tests. In absence of such tests, conversion factors of 1.05 for cubic specimen (200mm) and 1.25 for cylindrical specimen (150mm diameter and

300mm height) may be applied to obtain the equivalent characteristic strength on the basis of 150mm cubes.

- **Elastic deformation**

The elastic deformations of both Normal and HSC i.e. *Modulus of Elasticity (E_{cm})* largely depends not only on the concrete grade but also on the actual properties of aggregates used. Since the grade of concrete corresponds to strength at an age of 28 days, the values of E_{cm} are also of the same age. The modulus of elasticity of a concrete is controlled by the moduli of elasticity of its components. In the absence of a more accurate data, or in case where great accuracy is not required, an estimate of the mean value of the secant modulus E_{cm} can be obtained for a given concrete grade. Where, E_{cm} is in GPa and f_{ck} in MPa. These values represent concrete cured under normal conditions and made with aggregates predominantly consisting of quartzite gravel.

$$E_{cm} = 22\,[(f_{ck} + 8)/10]^{0.3}$$

- **Poisson's ratio**

Poisson's ratio is the ratio of transverse to longitudinal strain. It tends to decrease with increasing water-cement ratio. This value may be taken equal to 0.2 for uncracked concrete and 0 for cracked concrete.

- **Coefficient of Thermal Expansion**

The coefficient of thermal expansion of concrete may be taken as 10×10^{-6} strain per ^{o}C.

- **Creep and shrinkage**

Creep and shrinkage of concrete depends on the ambient humidity, the dimensions of the element and the composition of the concrete. Long term deflection, creep, is also influenced by the maturity of the concrete when the load is first applied and depends on the duration and magnitude of the loading. Any estimation of the creep coefficient $\phi(t,t_0)$ and of the basic shrinkage strain, ϵ_{cs} shall take these parameters into account. In cases where great accuracy is not required, the creep coefficient, $\phi(t,t_0)$ can be related to E_c, the tangent modulus, which may be taken as $1.05\ E_{cm}$. The values given below in Tables 1-1 and 1-2 respectively can be considered as the final creep coefficient $\phi(\infty,t_0)$ and the final shrinkage strain $\epsilon_{cs\infty}$. These values apply for a range of the mean temperature of the concrete between 10ºC and 20ºC. Maximum seasonal temperature up to 40ºC can be accepted. In the same way, variations in relative humidity around the mean values given in Tables 1-1 and 1-2 between RH=20% and RH=100% are acceptable.

Age at Loading t_o (days)	*Notional size 2Ac/u in (mm)*					
	50	*150*	*600*	*50*	*150*	*600*
	Dry atmospheric conditions (Inside)			*Humid atmospheric conditions(outside)*		
	(RH=50%)			*(RH=80%)*		
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2
28	3	2.5	2	1.9	1.7	1.5
90	2.4	2	1.6	1.5	1.4	1.2
365	1.8	1.5	1.2	1.1	1	1

Table 1-1 Final Creep Coefficient $\phi\ (\infty, t_0)$

Location of the number	*Relative humidity (%)*	*Notional size 2Ac/u (mm)*	
		< or = 150	*600*
Inside	50	-0.6	-0.5
Outside	80	-0.33	-0.28

Table 1-2 Final Shrinkage Strains $\epsilon_{cs\infty}$ (in %)

- **Stress-Strain relation**

The relation between stress σ_c and strain ε_c for short term uniaxial loading is described by the following expression and other idealized stress-strain relations may be applied, if they represent the behavior of the concrete considered adequately.

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta}$$

where:

$\eta = \varepsilon_c / \varepsilon_{c1}$

ε_{c1} is the strain at peak stress

$k = 1,05\, E_{cm} \times |\varepsilon_{c1}| / f_{cm}$

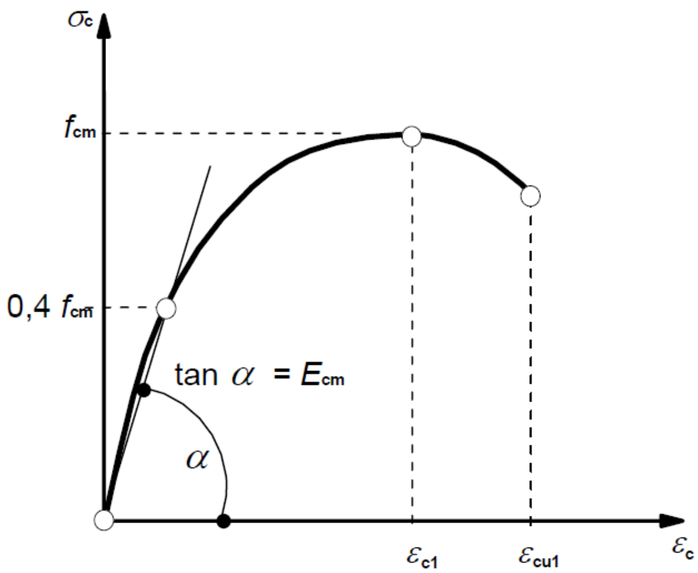

Figure 1-1 Schematic representation of the Stress-Strain relation for structural analysis

- **Design Compressive and tensile strength**

The value of the design compressive strength, f_{cd} , is defined as:

$$f_{cd} = (\alpha_{cc} f_{ck}) / \gamma_C$$

Where:

γ_C is the partial safety factor for concrete

α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied.

Note: - The value of α_{cc} for use in should lie between 0.8 and 1.0

The value of the design tensile strength, f_{ctd}, is defined as:

$$f_{ctd} = (\alpha_{ct} f_{ctk}) / \gamma_C$$

Where:

γ_C is the partial safety factor for concrete

f_{ctk} is the characteristic tensile strength of concrete

α_{ct} is a coefficient taking account of long term effects on the tensile strength and of unfavorable effects, resulting from the way the load is applied.

- **Stress-Strain relations for the design of cross-sections**

For the design of cross-sections, the following stress-strain relationship may be used.

$$\sigma_c = f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \quad \text{for} \quad 0 \le \varepsilon_c \le \varepsilon_{c2}$$

$$\sigma_c = f_{cd} \quad \textit{for} \quad \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2}$$

where:

n is the exponent

ε_{c2} is the strain at reaching the maximum strength

ε_{cu2} is the ultimate strain according to Table 1-3

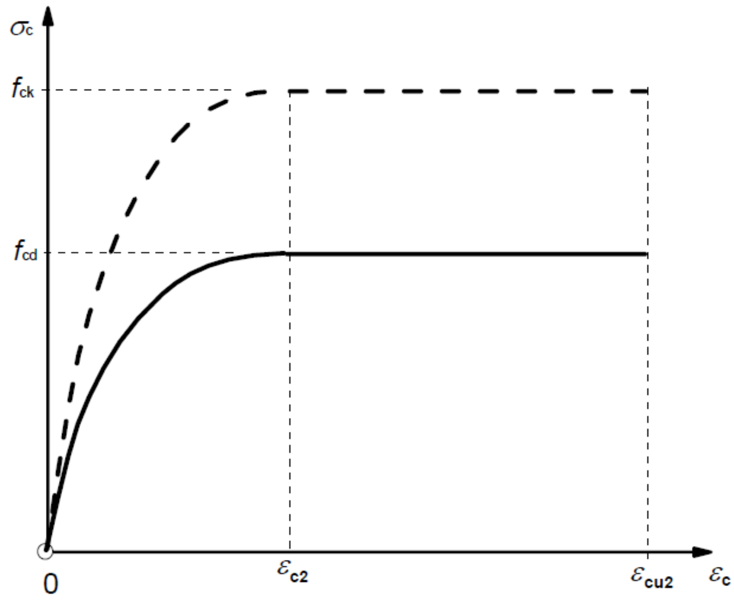

Figure 1-2 Parabola-rectangle diagram for concrete under compression

Other simplified stress-strain relationships may be used if equivalent to or more conservative than the one defined in Figure 1-2, for instance bi-linear according to Figure 1-3 (compressive stress and shortening strain shown as absolute values) with values of ε_{c3} and ε_{cu3}.

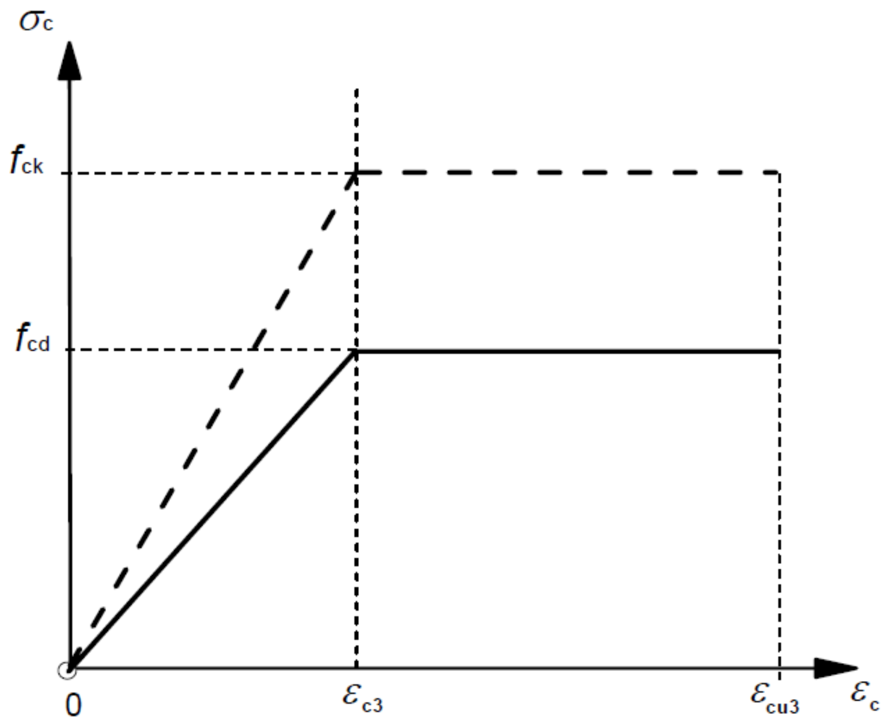

Figure 1-3 Bi-linear stress-strain diagram for concrete under compression

Also, rectangular stress distribution may be assumed for the stress-strain relation for the design of cross-sections. The factor λ, defining the effective height of the compression zone and the factor η, defining the effective strength, follow from:

$\lambda = 0{,}8$ for $f_{ck} \leq 50$ MPa
$\lambda = 0{,}8 - (f_{ck} - 50)/400$ for $50 < f_{ck} \leq 90$ MPa

and

$\eta = 1{,}0$ for $f_{ck} \leq 50$ MPa
$\eta = 1{,}0 - (f_{ck} - 50)/200$ for $50 < f_{ck} \leq 90$ MPa

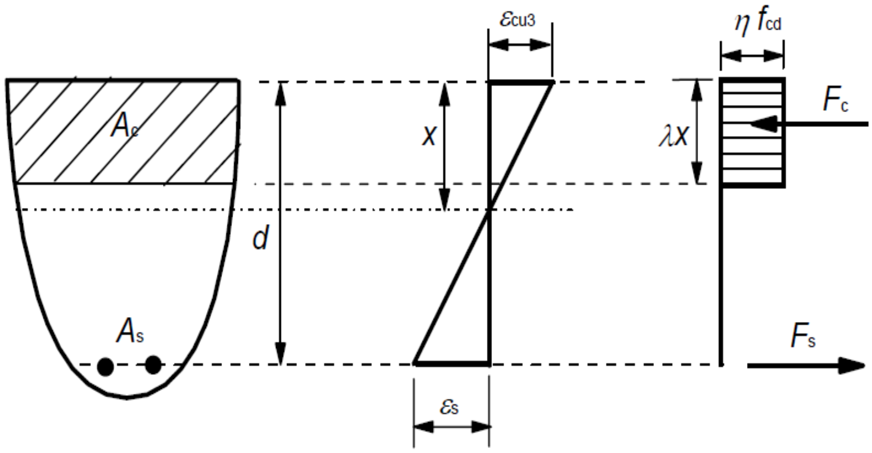

Figure 1-4. Rectangular stress distribution

Generally, much research in recent years has been devoted to establishing the fundamental and engineering properties of HSC, as well as the engineering characteristics of structural members made with the material. A large body of information is now available, permitting the engineering to use HSC with confidence when its advantages justify the higher cost. Some shortcomings of HSC include brittle behavior in compression, somewhat reduced ultimate strain capacity, an increased tendency to crack when drying and lower creep exhibited by the material.

HSC exhibits improved durability and abrasion resistance. As broader experience is gained in practical applications, and as design codes are gradually updated to recognize the special properties of HSC now available, much wider use can be expected. According to ACI Committee Report 363R-92, an essential requirement for HSC is a low water-cementious material ratio. Water-cementitious ratios by weight for high-strength concretes typically have ranged from 0.27 to 0.50.

	Strength classes for concrete														Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$ (MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$ C50/60 $f_{ctm}=2,12 \cdot \ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0,7 f_{cm}^{0,31} < 2,8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2,6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3}(‰)$=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2,6+35[(90-f_{ck})/100]^4$

Table 1-3 Strength and deformation characteristics of concrete

CHAPTER 2—STRUCTURAL ANALYSIS

2.1—INTRODUCTION

The purpose of a structural analysis is to establish the distribution of either internal forces and moments, or stresses, strains and displacements, over the whole or part of a structure. Analysis will be used to establish the distribution of internal forces and moments, and the complete verification or demonstration of resistance of cross sections. The following considerations are taken for establishing cross-sections of columns, beams, walls and slabs:-

a) The Ultimate Limit State (ULS) is the critical limit state.

b) The calculation of the Ultimate resistance of members for flexure and axial loads shall be based on the following assumptions:-

   1) Plane sections remain plane
   2) The reinforcement is subjected to the same variations in strain as the adjacent concrete
   3) The tensile strength of concrete is neglected
   4) The maximum compressive strain in the concrete is taken to be 0.0035 in bending and 0.002 in axial compression.
   5) The maximum tensile strain in the reinforcement is taken to be 0.01

c) Linear analysis of slabs shall be based generally on the gross cross-sections by adopting poission's ratio value between 0 and 0.2. This value is valid both for Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

2.2—STRUCTURAL SYSTEM

Structural system in buildings is defined as the particular method of assembling and constructing structural elements of a building so that they support and transmit applied loads safely to the ground without exceeding the allowable stresses in the members. Concrete buildings shall be classified into one of the following structural types according to their behavior under horizontal seismic actions.

a) *Frame System: -* Structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system.

b) *Dual System: -* Structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls. There are two types of Dual Systems defined as *Frame Equivalent* and *Wall Equivalent*. System in which the shear resistance of the frame system at the building base is greater than 50% of the total shear resistance of the whole structural system is defined as *Frame Equivalent*. Whereas, it is defined as *Wall Equivalent* if the shear resistance of the walls at the base is higher than 50% of the total seismic resistance of the whole building.

c) *Ductile Wall System: -* Wall fixed at the base so that the relative rotation of the base with respect to the rest of the structural system is prevented, and that is designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large perforations, just above its base.

d) *System of Large Lightly Reinforced Walls:* - Wall with large cross-sectional dimensions, that is, a horizontal dimension at least equal to 4.0 m or two-thirds of the height of the wall, whichever is less, which is expected to develop limited cracking and inelastic behavior under the seismic design situation.

e) *Inverted Pendulum System:* - system in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element.

f) *Torsionally Flexible System:* - dual or wall system not having a minimum torsional rigidity. An example of this is a structural system consisting of flexible frames combined with walls concentrated near the center of the building in plan.

2.3— ANALYSIS OF BUILDINGS WITH NSC AND HSC

In this thesis, ten buildings were analyzed. Five out of these ten buildings had columns and shear walls with HSC (C95), whereas the rest five had the same structural elements analyzed with NSC (C30). In addition, different building heights were taken for comparison of these buildings. Two out of these eight buildings were G+25; two G+20, two G+15, the other two G+10 and the remaining two were G+5. Out of these ten buildings, this thesis as a sample covers the analysis of two G+25 buildings having HSC and NSC columns and shear walls. These two buildings were chosen randomly without any pre-conditions.

The Structural Analysis of a G+25 apartment building to be constructed in Addis Ababa, Ethiopia with NSC and HSC having a compressive strength of 30 MPa and 95 MPa respectively is presented in the following pages.

The whole structure has been modeled as a 3D Wall Equivalent Dual System since the shear resistance of the walls at the building base is higher than 50% of the total seismic resistance of the whole structural system. The full 3D load and displacement effect, the actual effect of monolithic construction such as T-beam effect and load transfer paths can be realized more reasonably by narrowing the behavioral gap that exists between the model and the real structure by using Extended 3D Analysis of Building Systems Software (ETABS). The self-weight of structural members have been considered by the software and the sizing of the structural members had been carried out according to EBCS-2 and Eurocode-2.

This Wall Equivalent Dual Structural System consists of two way solid slab elements which act as diaphragm. Square reinforced concrete columns were used to take up the vertical loads whereas Shear walls were used mainly to resist lateral loadings so as to ignore global second order effects in buildings. In addition, beams were used to support the floor and roof system.

2.3.1—Considered Loadings

The major loadings considered are:

A) Vertical Loadings (Gravity Loads):

- *Dead Load (DL)* – Self Weight, Wall Load and Finishing Load, Roof Load, partition load.
- *Live Load (LL)* – Expected service loads from live load on the respective purpose of the structure.

B) Lateral Loadings:

- Static Seismic Load (Earth Quake load) and
- Wind Load (WL)

These loadings make up for the following twelve load combinations.

No.	*Combination Name*	*Vertical Loading*	*Lateral loading*	*Eccentricity for Earth Quake*
1	Comb 1	1.3*DL + 1.6*LL	-	-
2	Comb 2	(0.75xComb1)	+EQxP	5%
3	Comb 3	(0.75xComb1)	-EQxP	5%
4	Comb 4	(0.75xComb1)	+EQxN	5%
5	Comb 5	(0.75xComb1)	-EQxN	5%
6	Comb 6	(0.75xComb1)	+EQyP	5%
7	Comb 7	(0.75xComb1)	-EQyP	5%
8	Comb 8	(0.75xComb1)	+EQyN	5%

9	Comb 9	(0.75xComb1)	-EQyN	5%
10	Comb 10	1*DL+1*LL	-	-
11	Comb 11	(0.7xComb 1)	+WIND X	-
12	Comb 12	(0.7xComb 1)	+WIND Y	-

Table 2-1 Load Combinations

2.3.2—Analysis codes used

The codes used for analysis are:

- Ethiopian Building Code Standard (EBCS-1) :- Basis of Design and Actions on Structures
- Ethiopian Building Code Standard (EBCS-2) :- Structural use of Concrete which is almost similar to Euro Code 2-1992
- Ethiopian Building Code Standard (EBCS-8):- Design of Structures for Earthquake Resistance which is almost similar to Eurocode 8-1998

2.3.3—Software used

ETABS Nonlinear Version 9.7.4 was used for the analysis of the building by modeling the structures as a 3D space frame system. ETABS is a powerful program that can greatly enhance an engineer's analysis and design capabilities for structures. Part of that power lies in an array of options and features. The other part lies in how simple it is to use. The basic approach for using the program is very straightforward. The user establishes grid lines, places structural objects relative to the grid lines using points, lines and areas, and assigns loads and structural properties to those structural objects (for example, a line object can be assigned section properties; a point object can be assigned spring properties; an area object can be assigned slab or deck properties). Analysis and design are then performed based on the structural objects and their assignments. Results are generated in graphical or tabular form that can be printed to a printer or to a file for use in other programs.

2.3.4— Materials

Steel Strength: - According to Eurocode 2,1992, the application rules for analysis, design and detailing are valid for a specified yield strength range, f_{yk} = 400 MPa to 600 MPa. In this analysis, the minimum yield strength i.e. 400 MPa has been used.

Concrete Strength: - Concrete strength is classified according to the minimum 28 days crushing compressive strength of 150mm cubes in N/mm^2. All Structural concrete members are to be C30 for the NSC model. Whereas, all slabs and beams are to be C45 and columns and shear walls are to be C95 concrete for the HSC model.

2.3.5 — Bearing Capacity recommendation

In this thesis, it is assumed that the allowable bearing capacity of soil to be taken as σ_{all} = 300 kPa at a depth of 3.0 m from NGL.

2.3.6—Structural System Layout and member sizing of Normal-Strength and High-Strength concrete models

A structural system layout shows how beams, columns, walls, roof and floor systems are modeled in 3D. Serviceability limit state according to Ethiopian Building Code Standard (EBCS-2) and Eurocode 2, 1992 were used to obtain initial member sizes of solid slabs and beams which are shown in Appendix-C of the document. After several iterations, shear wall sizes for both models were obtained so as to neglect second order moments according to Eurocode 2, 1992. These calculations are shown in Appendix-E of this thesis. Also, preliminary column sizes were assumed at the beginning of the modeling process based on Eurocode 2, 1992 and were increased according to the factored axial compression and biaxial bending moment demands of sections. Span lengths of 6.0 m between columns and floor to floor height of 3.0 m have been assumed for analysis.

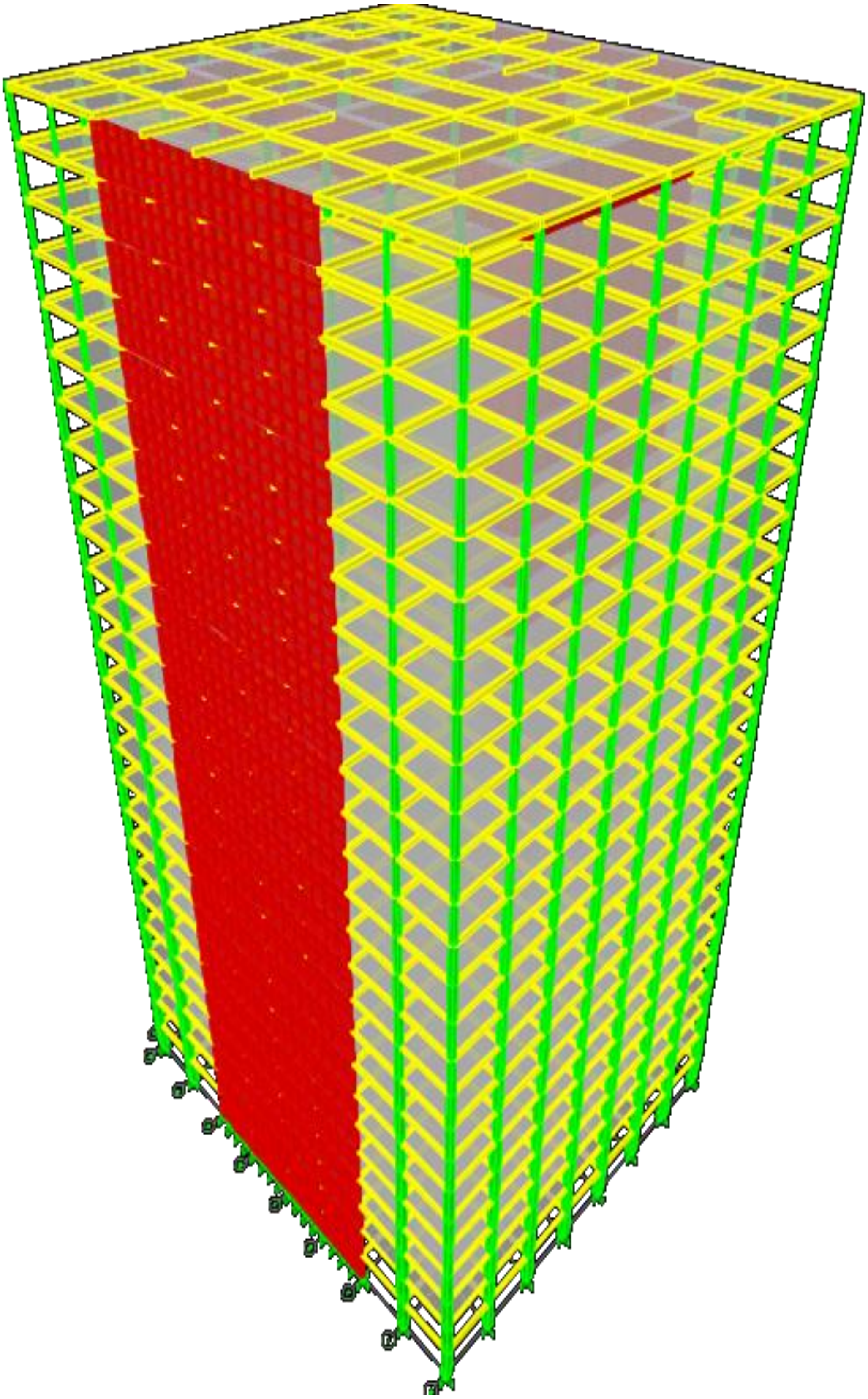

Figure 2-1 3D Structural System Layout

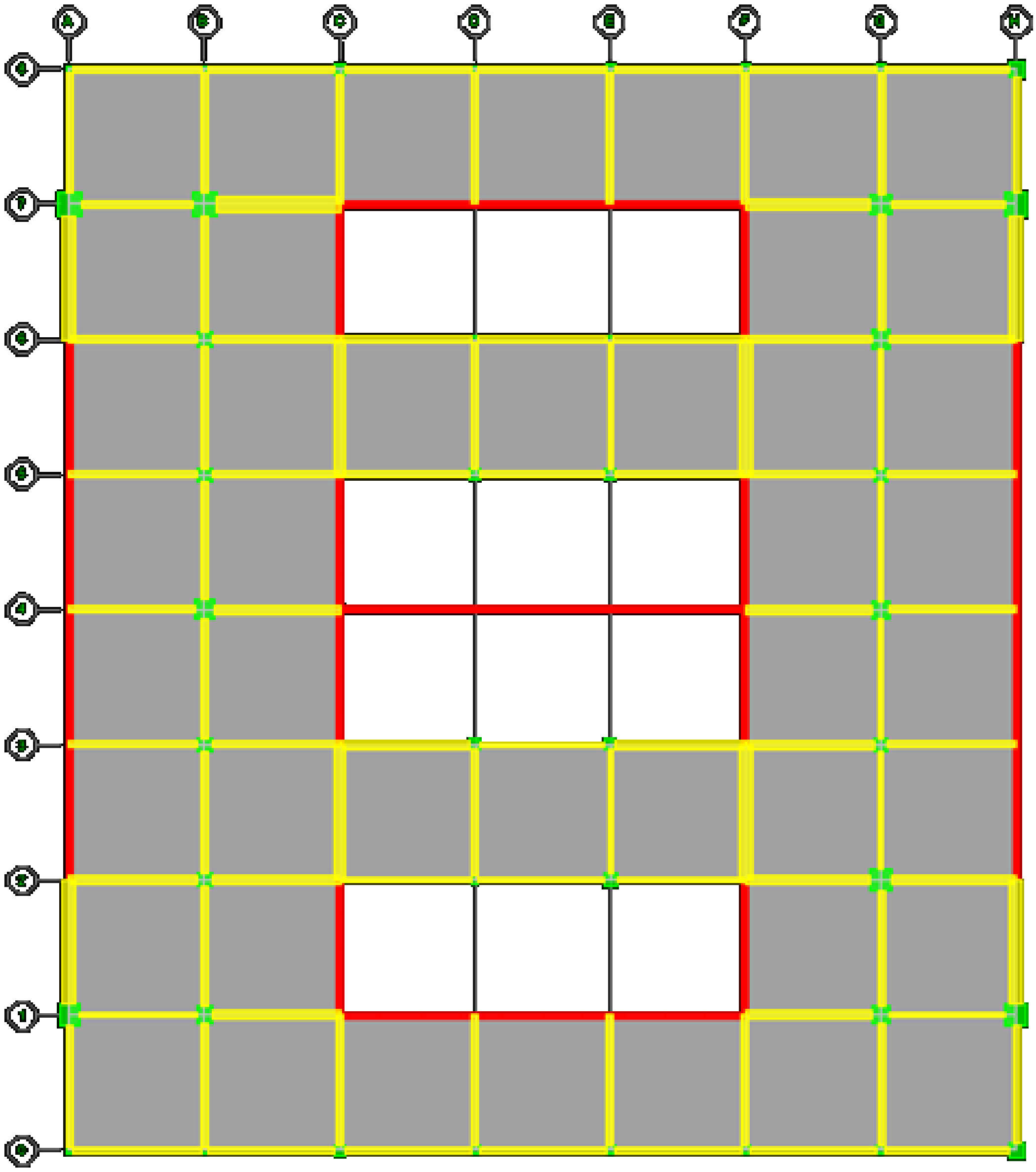

Figure 2-2 Typical Floor Structural System Layout

2.3.7 – Analysis of Shear Wall

Shear walls are concrete walls which contribute to the lateral stability of the structure by efficiently resisting lateral loads. Shear walls may be described further as vertical cantilevered beams, which resist lateral wind and seismic loads acting on a building. Lateral load resisted by each shear wall in a structure should be obtained from a global analysis of the structure, taking into account the applied loads, the eccentricities of the loads with respect to the shear-center of the structure and the interaction between the different structural walls. Due to the low tensile strength of concrete, reinforced concrete shear walls tend to behave in a nonlinear manner with a significant reduction in stiffness, even under service loads. To accurately assess the lateral deflection of shear walls, the prediction of stiffness of these members after cracking becomes important.

According to EBCS-2, the thickness of load bearing walls shall not be less than 1/25 of the unsupported height or width, whichever is shorter, nor less than 150 mm. After so many iterations, shear walls with thickness of 350mm for Normal-Strength and 300mm for High-Strength concrete models were used so as to neglect second order moments.

Appendix-E shows the analysis of second order effects using Normal-Strength Concrete model. Despite the difference in shear wall thickness, the same analysis method has been used for the High-Strength Concrete model.

2.3.7.1— Global second order effects in buildings

According to Eurocode 2, 1992, global second order effects in buildings may be ignored if:

$$W \leq k_1 \frac{n_s}{n_s + 1.6} \frac{\sum E_{cd} I_c}{L^2}$$

Where:

W is the total vertical load of building

n_s is the number of stories

L is the total height of building above level of moment restraint

E_{cd} is the design value of the modulus of elasticity of concrete

I_c is the second moment of area (uncracked concrete section) of bracing member(s)

k_1 is the relative flexibilities of rotational restraints where the recommended value is 0.31

The above expression is valid only if all the following conditions are met:

- Torsional instability is not governing, i.e. structure is reasonably symmetrical
- Global shear deformations are negligible (as in a bracing system mainly consisting of shear walls without large openings)
- Bracing members are rigidly fixed at the base, i.e. rotations are negligible
- The stiffness of bracing members is reasonably constant along the height
- The total vertical load increases by approximately the same amount per story

The second order analysis for these buildings uses, two U-type, two I-type and one H-type of walls for lateral resistance. These Analyses are shown in Appendix E.

2.3.7.2— Cracking of shear wall

The structural model for the analysis of the building shall represent the stiffness properties of the entire system. Uncracked elastic stiffness may be used for analysis or, preferably and more realistically, cracked stiffness in order to account for the influence of cracking on deformations. In the absence of an accurate evaluation of the stiffness properties, substantiated by rational analysis, the cracked bending and shear stiffness may be taken as one half of the gross section uncracked elastic stiffness.

In this analysis, the shear wall acts as a cantilever beam. The characteristic tensile strength (f_{ctm}) refers to the highest stress reached under concentric tensile loading. The stresses of both models are greater than the characteristic tensile strength of the Normal-Strength Concrete which is equal to 2.6 MPa and 4.8 MPa for the High-Strength Concrete which indicate the cracking of both shear walls.

2.3.8 —Lateral Analysis of Structural Models

2.3.8.1— Introduction

Lateral loads can be classified into two main parts. The first one is Seismic load (Earthquake Load) and the second one being Wind load. The analysis of the two models for wind force, which was found to be insignificant, has been shown in Appendix -D of the thesis. On the other hand, the analysis of Earthquake force for the two high-rise structures has been discussed in the coming few pages of the thesis. The seismic analysis of the two buildings is shown in Appendix – F of the document.

2.3.8.2 —Characteristics of Earthquake Resistant Buildings

In seismic regions the aspect of seismic hazard shall be taken into account in the early stages of the analysis of a building, thus enabling the achievement of a structural system which, within acceptable costs, satisfies the fundamental requirements. The guiding principles governing this early stage of analysis are:

a) Structural Simplicity:-

Structural simplicity is characterized by the existence of clear and direct paths for the transmission of the seismic forces.

b) Uniformity and Symmetry:-

Uniformity and symmetry in plan is characterized by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building.

c) Bi-directional resistance and Stiffness:-

Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction by arranging structural elements so as to ensure similar resistance and stiffness in both directions.

d) Torsional resistance and stiffness:-

Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

e) *Diaphragmatic behavior at story level:-*

In buildings, floors (including the roof) play a very important role in the overall seismic behavior of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action.

f) *Adequate foundation:-*

With regard to the seismic action, the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

2.3.8.3—Fundamental requirements of structures in seismic regions

Structures in seismic regions shall be designed and constructed in such a way that the Ultimate Limit States (ULS) and Serviceability Limit States (SLS) are met.

ULS are those associated with collapse or with other forms of structural failure which might endanger the safety of people. The no-collapse requirement (ULS) under the seismic design situation is considered to have been met if conditions regarding those associated with damage beyond which specified service requirements are no longer met. An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits.

For most building structures, especially High-Rise buildings, the P-Delta effect of most concern occurs in the columns due to gravity load, including dead and live load. The column axial forces are compressive, making the structure more flexible against lateral loads.

The two types of P-Delta effects are first due to the overall sway of the structure and the second due to the deformation of the member between its ends. The former effect is often significant; it can be accounted for accurately by considering the total vertical load at a story level, which is due to gravity loads and is unaffected by any lateral loads.

Accordingly, global second-order effects (P-Delta effects) of the whole structure need not be taken into account if the inter-story drift sensitivity coefficient θ, is less than 0.1 in all stories. If $0.1 < \theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.The value of the coefficient θ shall not exceed 0.3.

$$\theta = \frac{P_{tot} \; d_r}{V_{tot} \; h} \leq 0.10$$

Where:

θ is the inter-story drift sensitivity coefficient

P_{tot} is the total gravity load at and above the story considered in the seismic design situation

d_r is the design inter-story drift, evaluated as the difference of the average lateral displacements

V_{tot} is the total seismic story shear

h is the inter-story height.

From the analysis output, Inter story drift sensitivity coefficients θx and θy in X and Y directions are calculated as follows for the Normal-Strength Concrete (NSC) model:

Floor	*Displacements(m)*		*h(m)*	P_{tot}	V_{tot}	*Avg. Displacements*		*θx*	*θy*
	x	*y*				*drx*	*dry*		
25th	-0.0003	-0.0004	3	15925.7	2570	0.0001	0	0.00027	0
24th	-0.0002	-0.0004	3	44039.5	760.6	0	0	0	0
23rd	-0.0002	-0.0004	3	72441.6	257.7	0	0	0	0
22nd	-0.0002	-0.0004	3	100967	250.3	0	0.0001	0	0.01344
21st	-0.0002	-0.0003	3	130199	257.9	0	0	0	0
20th	-0.0002	-0.0003	3	159634	253.1	0	0	0	0
19th	-0.0002	-0.0003	3	189776	255.7	0.0001	0	0.02431	0
18th	-0.0001	-0.0003	3	220583	260.1	0	0.0001	0	0.02827
17th	-0.0001	-0.0002	3	252062	260.2	0	0	0	0
16th	-0.0001	-0.0002	3	283930	252.7	0	0	0	0
15th	-0.0001	-0.0002	3	315903	239.3	0	0	0	0
14th	-0.0001	-0.0002	3	348373	232.2	0	0	0	0
13th	-0.0001	-0.0002	3	381536	226.9	0	0.0001	0	0.05638
12th	-0.0001	-0.0001	3	415728	224.5	0	0	0	0
11th	-0.0001	-0.0001	3	450115	209.7	0.0001	0	0.07156	0
10th	0	-0.0001	3	484461	191.9	0	0	0	0
9th	0	-0.0001	3	518220	165.3	0	0	0	0
8th	0	-0.0001	3	551776	145.7	0	0.0001	0	0.12622

7th	0	0	3	584516	120.9	0	0	0	0
6th	0	0	3	616581	98.59	0	0	0	0
5th	0	0	3	648312	80.12	0	0	0	0
4th	0	0	3	679562	61.69	0	0	0	0
3rd	0	0	3	710180	43.63	0	0	0	0
2nd	0	0	3	741229	29.62	0	0	0	0
1st	0	0	3	772294	15.75	0	0	0	0

Table 2-2 Inter story drift coefficients for Normal-Strength Concrete (NSC)

Similarly, Inter story drift sensitivity coefficients θx and θy in X and Y directions are calculated as follows for the High-Strength Concrete (HSC) model:

Floor	*Displacements(m)*		*h*	P_{tot}	V_{tot}	*Avg. Displacements*		θx	θy
	x	*y*	*(m)*			*drx*	*dry*		
25th	0.0001	-0.0002	3	27264.1	2552.7	0	0	0.00000	0
24th	0.0001	-0.0002	3	54974.0	833.5	0.0001	0	0.00223	0
23rd	0	-0.0002	3	82427.4	768.7	0	0	0	0
22nd	0	-0.0002	3	109989.4	746.4	0	0	0	0.0000
21st	0	-0.0002	3	137748.3	732.7	0	0.0001	0	0.00626
20th	0	-0.0001	3	165670.8	713.6	0	0	0	0
19th	0	-0.0001	3	194133.4	725.5	0	0	0.00000	0
18th	0	-0.0001	3	223162.3	738.2	0	0	0	0.00000
17th	0	-0.0001	3	252667.8	736.8	0	0	0	0
16th	0	-0.0001	3	282389.7	713.1	0	0	0	0
15th	0	-0.0001	3	312606.7	701.5	0	0	0	0
14th	0	-0.0001	3	342911.2	662.6	0	0	0	0

13th	0	-0.0001	3	373366.1	621.7	0	0	0	0.00000
12th	0	-0.0001	3	404024.6	589.9	0	0.0001	0	0.02282
11th	0	0	3	434835.1	549.4	0	0	0.00000	0
10th	0	0	3	465906.5	508.9	0	0	0	0
9th	0	0	3	497116.8	366.7	0	0	0	0
8th	0	0	3	528506.1	89.02	0	0	0	0.00000
7th	0	0	3	559849.2	369.3	0	0	0	0
6th	0	0	3	590588.2	315.5	0	0	0	0
5th	0	0	3	621103.3	250.0	0	0	0	0
4th	0	0	3	651049.7	196.6	0	0	0	0
3rd	0	0	3	680535.4	139.8	0	0	0	0
2nd	0	0	3	709557.1	89.06	0	0	0	0
1st	0	0	3	738542.8	42.21	0	0	0	0

Table 2-3 Inter story drift coefficients for High-Strength Concrete (NSC)

Also, unless otherwise specified, the following limits on design inter story drift shall be observed to satisfy Serviceability Limit State (SLS) requirements.

a) For buildings having non-structural elements of brittle materials attached to the structure:

$$d_r \, v \leq 0.005 \, h$$

b) For buildings having ductile non-structural elements:

$$d_r \, v \leq 0.0075 \, h$$

c) For buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r \, v \leq 0.010 \, h$$

Where:

d_r is the design inter-story drift, evaluated as the difference of the avg. lateral displacements at the top and bottom of the story under consideration

h is the story height

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

The value of the reduction factor v may also depend on the importance class of the building. Values of v are 0.4 for importance classes III and IV and v = 0.5 for importance classes I and II.

In this analysis, for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r\ v \leq 0.005\ h$$

Where:

h = 3.0 m for all floors

v = 0.5

Floor	*Displacements(m)*		*h(m)*	*Red. Fac.*	*Avg. Displacements*		*(v)(drx)*	*(v)(dry)*	*(0.005)(h)*
	x	*y*		*v*	*drx*	*dry*			
25th	-0.0003	-0.0004	3	0.5	0.0001	0.0000	0.0001	0.0000	0.0150
24th	-0.0002	-0.0004	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
23rd	-0.0002	-0.0004	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
22nd	-0.0002	-0.0004	3	0.5	0.0000	0.0001	0.0000	0.0001	0.0150

21st	-0.0002	-0.0003	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
20th	-0.0002	-0.0003	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
19th	-0.0002	-0.0003	3	0.5	0.0001	0.0000	0.0001	0.0000	0.0150
18th	-0.0001	-0.0003	3	0.5	0.0000	0.0001	0.0000	0.0001	0.0150
17th	-0.0001	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
16th	-0.0001	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
15th	-0.0001	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
14th	-0.0001	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
13th	-0.0001	-0.0002	3	0.5	0.0000	0.0001	0.0000	0.0001	0.0150
12th	-0.0001	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
11th	-0.0001	-0.0001	3	0.5	0.0001	0.0000	0.0001	0.0000	0.0150
10th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
9th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
8th	0	-0.0001	3	0.5	0.0000	0.0001	0.0000	0.0001	0.0150
7th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
6th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
5th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
4th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
3rd	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
2nd	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
1st	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150

Table 2-4 Design inter-story drift for Normal Strength Concrete (NSC)

Floor	*Displacements(m)*		*h(m)*	*Red. Fac.*	*Avg. Displacements*		*(v)(drx)*	*(v)(dry)*	*(0.005)(h)*
	x	*y*		*v*	*drx*	*dry*			
25th	0.0001	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
24th	0.0001	-0.0002	3	0.5	0.0001	0.0000	0.0044	0.0000	0.0150
23rd	0	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
22nd	0	-0.0002	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
21st	0	-0.0002	3	0.5	0.0000	0.0001	0.0000	0.0125	0.0150
20th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
19th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
18th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
17th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
16th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
15th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
14th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
13th	0	-0.0001	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
12th	0	-0.0001	3	0.5	0.0000	0.0001	0.0000	0.0457	0.0150
11th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
10th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150

9th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
8th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
7th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
6th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
5th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
4th	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
3rd	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
2nd	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150
1st	0	0	3	0.5	0.0000	0.0000	0.0000	0.0000	0.0150

Table 2-5 Design inter-story drift for High-Strength Concrete (HSC)

2.3.8.4 — Energy dissipation capacity and ductility classes

The analysis and design of earthquake resistant concrete buildings shall provide the structure with an adequate capacity to dissipate energy without substantial reduction of its overall resistance against horizontal and vertical loading. In the seismic design situation, adequate resistance of all structural elements shall be provided and non-linear deformation demands in critical regions should be proportionate with the overall ductility assumed in calculations.

Concrete buildings designed in accordance with Eurocode 8, are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their energy dissipation capacity. Both classes correspond to buildings designed, dimensioned and detailed in accordance with specific earthquake resistant provisions, enabling the structure to develop stable mechanisms associated with large energy dissipation without suffering brittle failures. To provide the appropriate amount of ductility in ductility classes M and H, specific provisions for all structural elements shall be satisfied in each class. In correspondence with the different available ductility in the two ductility classes, different values of the behavior factor γ are used for each class.

Concrete buildings may alternatively be designed for low dissipation capacity and low ductility (DCL), by applying only the rules of Eurocode-2 for the seismic design situation, and neglecting the specific provisions given in Eurocode 8. Design with this alternative, termed ductility class L (low), is recommended only in low seismicity cases.

Both of the buildings were analyzed for low dissipation capacity and low ductility thus, termed ductility class low (DCL) since Addis Ababa is found in a low seismic zone. Accordingly, the seismic analysis of these buildings is shown on Appendix F of this document. From Appendix F, the following Lateral forces per floor were summarized and shown on Tables 2-6 and 2-7 for both buildings.

Floor	*h (m)*	*Weight, W(kN)*	*Wihi*	*Lateral force Fi(kN)*
25th	75	5,837.8	437,832.9	2569.6
24th	72	9,299.4	669,557.0	760.6
23rd	69	9,665.8	666,943.3	257.7
22nd	66	9,813.3	647,676.5	250.3
21st	63	10,595.0	667,486.9	257.9
20th	60	10,916.0	654,957.6	253.1
19th	57	11,610.6	661,805.5	255.7
18th	54	12,465.6	673,140.3	260.1
17th	51	13,205.4	673,477.9	260.2
16th	48	13,625.2	654,010.4	252.7
15th	45	13,762.3	619,302.7	239.3

14th	42	14,310.7	601,051.3	232.2
13th	39	15,060.6	587,364.1	226.9
12th	36	16,141.1	581,079.6	224.5
11th	33	16,443.6	542,638.9	209.7
10th	30	16,555.3	496,659.7	191.9
9th	27	15,842.1	427,737.9	165.3
8th	24	15,713.2	377,117.1	145.7
7th	21	14,898.2	312,862.2	120.9
6th	18	14,175.4	255,156.8	98.6
5th	15	13,823.5	207,352.1	80.1
4th	12	13,304.0	159,647.6	61.7
3rd	9	12,547.3	112,925.4	43.6
2nd	6	12,775.2	76,651.4	29.6
1st	3	13,590.3	40,770.8	15.8
		Total	*11,805,206*	

Table 2-6 Lateral force per floor for Normal-Strength Concrete (NSC)

Floor	*Height, h (m)*	*Weight(kN)*	*Wihi*	*Lateral force Fi(kN)*
25th	75	5,479.5	410,965.2	2552.7
24th	72	8,204.4	590,715.1	833.5
23rd	69	7,895.7	544,805.9	768.7
22nd	66	8,015.0	528,988.5	746.4
21st	63	8,242.8	519,299.2	732.7
20th	60	8,429.3	505,757.0	713.6
19th	57	9,020.6	514,174.8	725.5
18th	54	9,689.0	523,207.8	738.3
17th	51	10,239.2	522,198.9	736.8

16th	48	10,529.4	505,411.2	713.1
15th	45	11,049.1	497,211.6	701.6
14th	42	11,181.2	469,610.3	662.6
13th	39	11,297.7	440,608.7	621.7
12th	36	11,613.7	418,094.1	589.9
11th	33	11,800.1	389,402.0	549.5
10th	30	12,023.2	360,696.5	508.9
9th	27	9,626.6	259,917.2	366.7
8th	24	2,628.8	63,092.0	89.0
7th	21	12,464.0	261,743.0	369.3
6th	18	12,422.2	223,599.2	315.5
5th	15	11,812.2	177,183.0	250.0
4th	12	11,615.8	139,389.9	196.7
3rd	9	11,015.4	99,138.4	139.9
2nd	6	10,520.6	63,123.8	89.1
1st	3	9,971.7	29,915.0	42.2
		Total	*9,058,248*	

Table 2-7 Lateral force per floor for High-Strength Concrete (HSC)

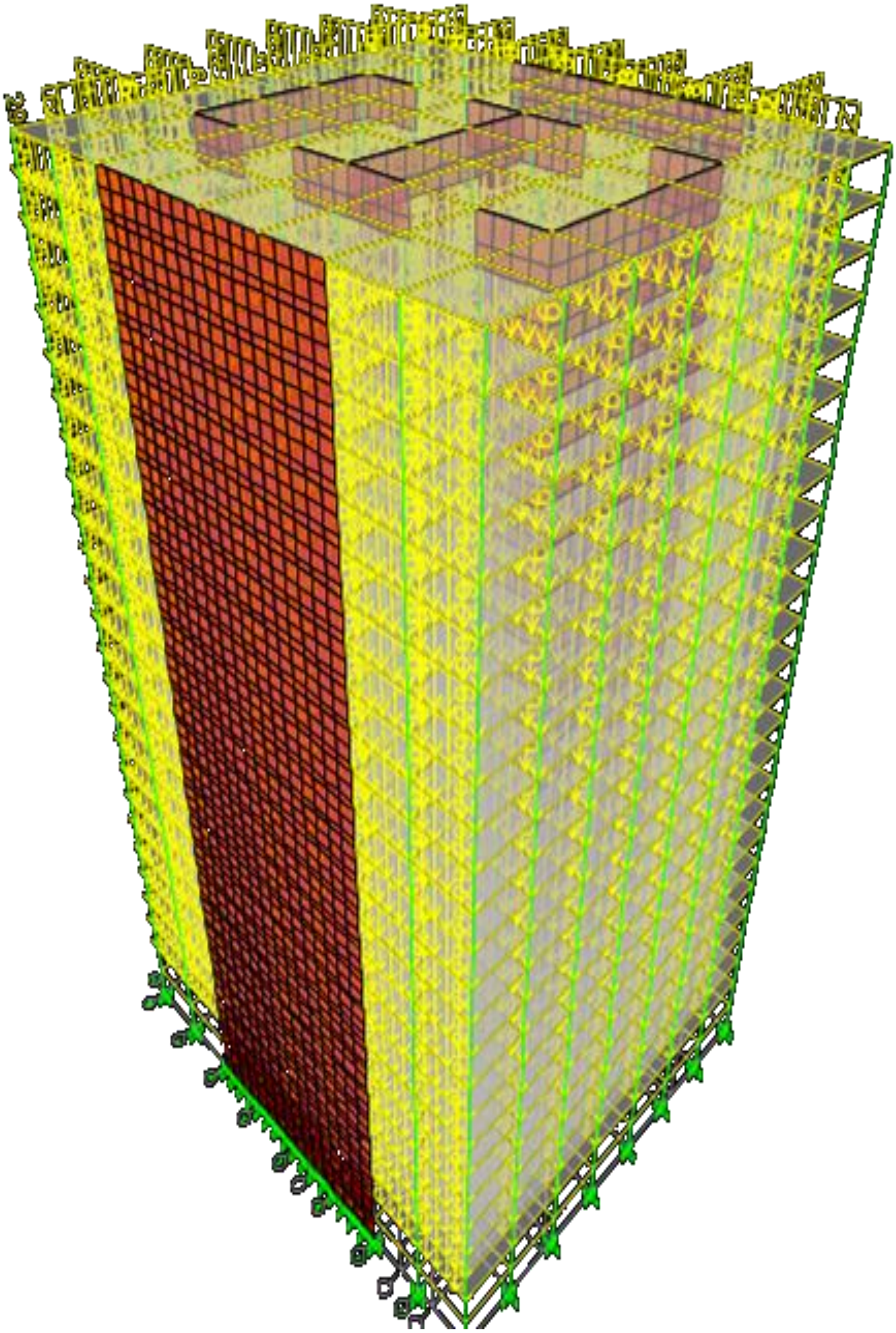

Figure 2-3 Building Loadings

CHAPTER 3—AREA SAVING AND ECONOMIC CONSIDERATIONS

3.1—INTRODUCTION

HSC is a state-of-the-art material that commands a premium price whose benefits are well worth the additional effort and expense. In many areas and for many uses, the benefits of HSC more than compensate for the increased costs of raw materials and quality control. The method and technology for producing HSC are not basically different from those required for concrete of normal grade except that the emphasis on quality control is perhaps greater with HSC.

Basically, HSC will carry a compression load at less cost than any lower-strength concrete. The reason for these economies is that, although the concrete itself is more costly than lower-Strength mixtures, the cost differential is offset by significant reduction in the given member size. This capability is particularly attractive for use in columns and shear walls.

3.2— COMPARISON OF BUILDINGS WITH DIFFERENT HEIGHTS

3.2.1— Area Saving

The area saving of the ten buildings that were mentioned in section 2.3 of the thesis is summarized and shown below in Table 3-1.

		(COL & SW) (m^2)		*Net area of building (m^2)*		
Building Type	*Total area of Building (m^2)*	*NSC*	*HSC*	*NSC*	*HSC*	*Area Saved (m^2)*
G+5 (H=15m)	11,520	298.6	243.2	11,221.4	11,276.8	55.4
G+10 (H=30m)	23,040	623.6	513.5	22,416.4	22,526.5	110.1
G+15(H=45m)	34,560	1,011.1	788.5	33,548.9	33,771.5	222.6
G+20 (H=60m)	46,080	1,404.9	1,059.5	44,675.1	45,020.5	345.3
G+25 (h=75m)	57,600	1,787.3	1,339.2	55,812.7	56,261.1	448.3

Table 3-1 Area saving of buildings with NSC and HSC

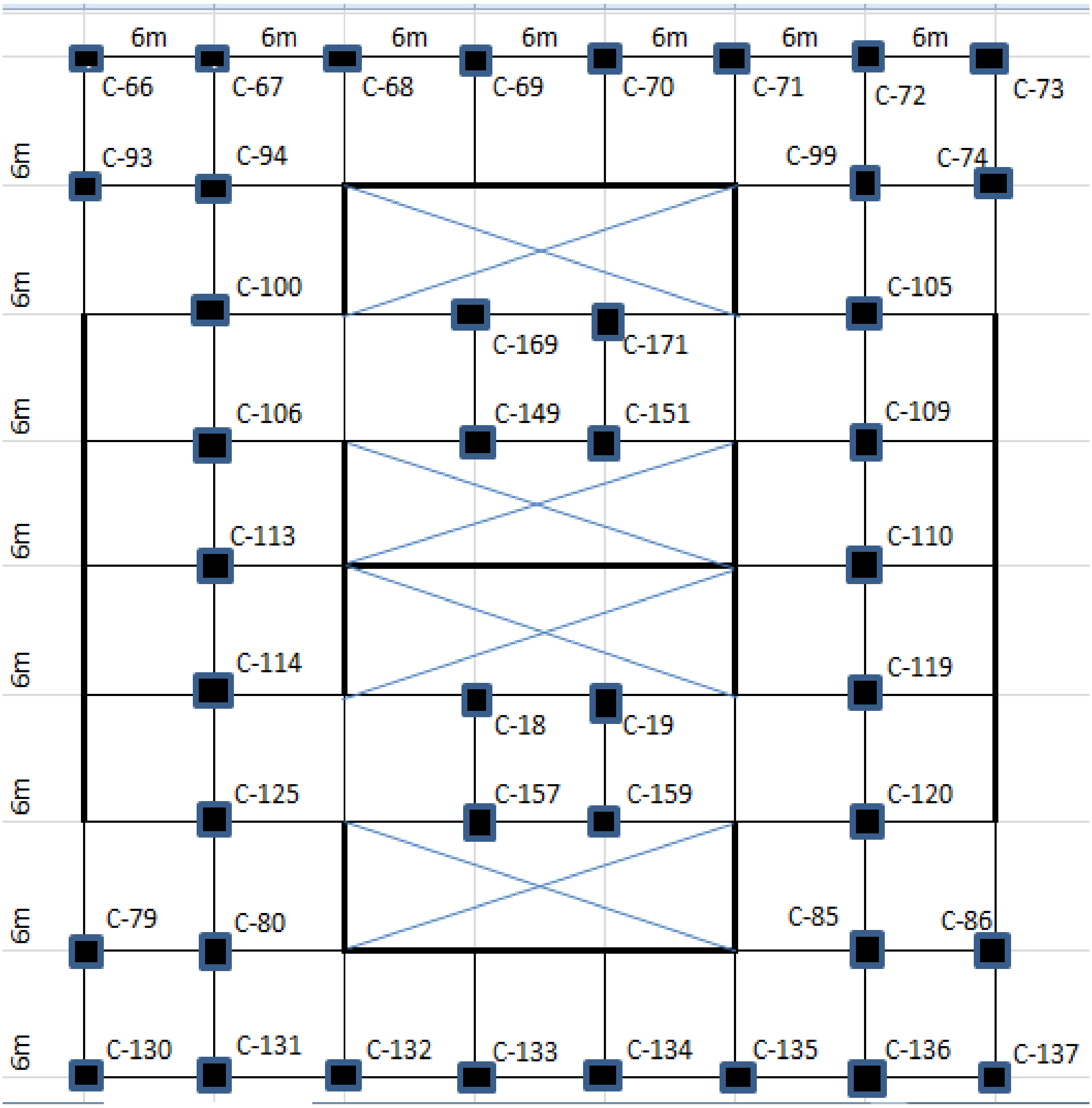

Figure 3-1 Column and shear wall layout of HSC building

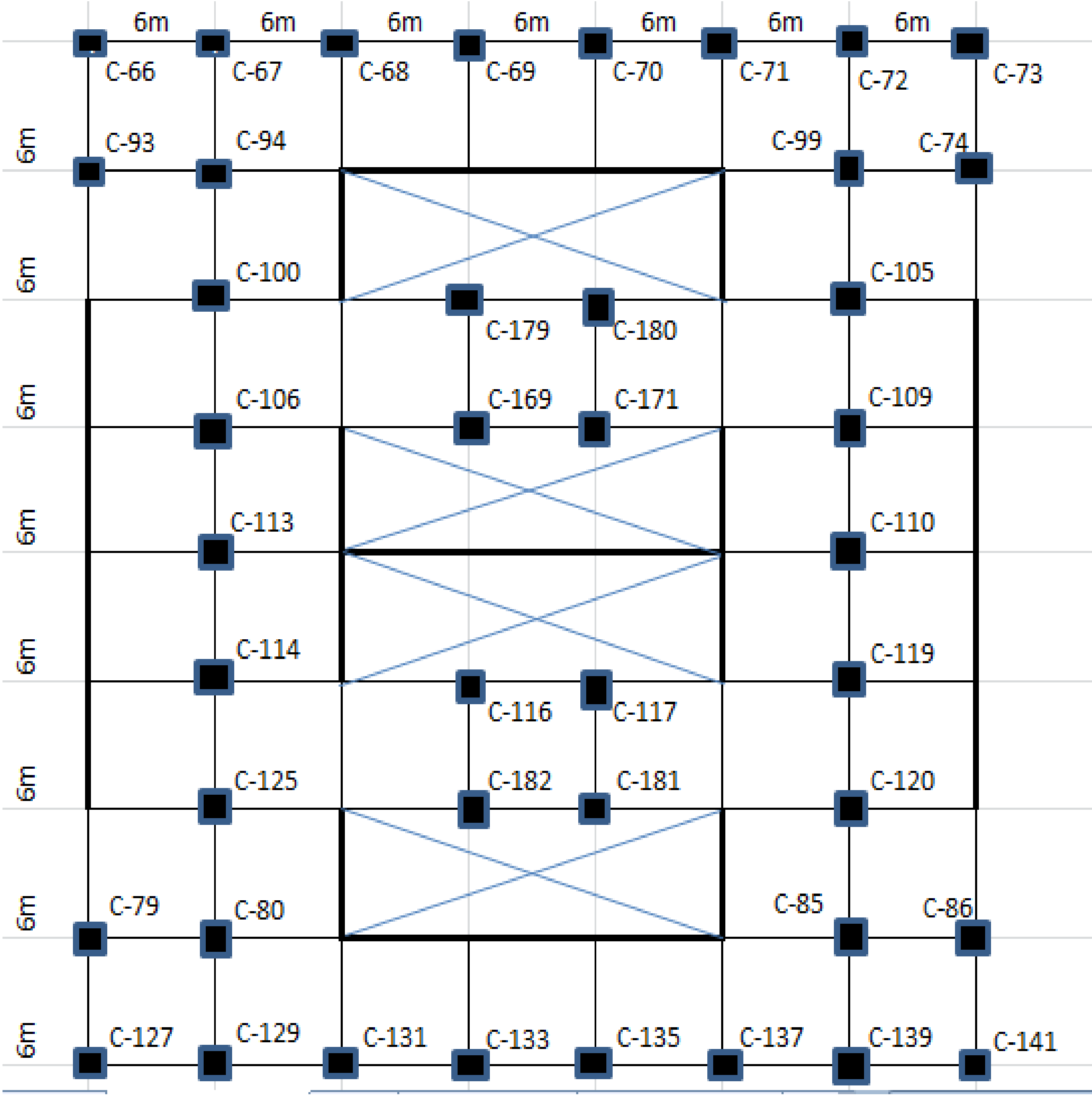

Figure 3-2 Column and shear wall layout of NSC building

3.2.2— Economic Considerations

Replacing percentages of steel with HSC in columns and shear walls has a high advantage in terms of cost. This idea is shown below by comparing the ten buildings defined in section 2.3 of the thesis.

	NSC (C30)				
	Columns				
	G+5	*G+10*	*G+15*	*G+20*	*G+25*
Conc. (m^3)	111.2	225.5	616.1	1,000.4	1,329.4
Unit Price of Conc.(Birr/m^3)	3,891.4	3,891.4	3,891.4	3,891.4	3,891.4
Rebar (kg)					
dia 6mm	1,042.2	4,117.2	7,035.5	12,048.3	16,062.6
dia 8mm					
dia 12mm					
dia 16mm	145,049.1	182,001.7	209,260.4	246,906.4	306,871.2
dia 20mm					
Unit Price of Rebar.(Birr/kg)					
dia 6mm	42.1	42.1	42.1	42.1	42.1
dia 8mm					
dia 12mm					
dia16mm	39.1	39.1	39.1	39.1	39.1
dia 20mm					
Cost of Conc.(Birr)	432,725.9	877,670.8	2,397,644.0	3,892,805.3	5,173,409.4
Cost of Rebar(Birr)	5,721,129.6	7,297,005.6	8,486,858.3	10,171,513.1	12,687,657.5
Total Cost (Birr)	6,153,855.5	8,174,676.4	10,884,502.2	14,064,318.5	17,861,066.9

Table 3-2 Bill of Quantities for NSC columns

	NSC (C30)				
	Shear Walls				
	G+5	G+10	G+15	G+20	G+25
Conc. (m^3)	693.3	1,708.2	2,550.0	3,416.5	4,193.0
Unit Price of Conc.(Birr/m^3)	3,891.4	3,891.4	3,891.4	3,891.4	3,891.4
Rebar (kg)					
dia 6mm					
dia 8mm	10,133.0	22,738.0	37,695.1	45,476.1	53,357.0
dia 12mm	9,001.8	16,081.8	23,508.2	32,163.6	37,018.9
dia 16mm	32,028.4	50,578.4	71,984.8	94,156.8	114,328.8
dia 20mm	32,158.9	50,777.9	72,293.1	94,555.7	114,818.4
Unit Price of Rebar.(Birr/kg)					
dia 6mm					
dia 8mm	42.1	42.1	42.1	42.1	42.1
dia 12mm	39.1	39.1	39.1	39.1	39.1
dia16mm	39.1	39.1	39.1	39.1	39.1
dia 20mm	39.1	39.1	39.1	39.1	39.1
Cost of Conc.(Birr)	2,697,921.5	6,647,498.8	9,923,179.4	13,294,997.5	16,316,587.9
Cost of Rebar(Birr)	3,291,523.0	5,554,479.0	8,155,244.0	10,560,998.1	12,665,673.1
Total Cost (Birr)	5,989,444.5	12,201,977.8	18,078,423.4	23,855,995.6	28,982,260.9

Table 3-3 Bill of Quantities for NSC shear walls

	HSC (C95)				
	Columns				
	G+5	*G+10*	*G+15*	*G+20*	*G+25*
Conc. (m^3)	105.8	194.3	317.9	446.7	599.2
Unit Price of Conc.(Birr/m^3)	4,936.5	4,936.5	4,936.5	4,936.5	4,936.5
Rebar (kg)					
dia 6mm	1,158.0	3,474.3	5,973.6	8,094.6	8,745.1
dia 8mm					
dia 12mm					
dia 16mm	5,241.3	32,770.6	97,388.1	147,569.5	232,042.6
dia 20mm					
Unit Price of Rebar.(Birr/kg)					
dia 6mm	42.1	42.1	42.1	42.1	42.1
dia 8mm					
dia 12mm					
dia16mm	39.1	39.1	39.1	39.1	39.1
dia 20mm					
Cost of Conc.(Birr)	522,281.7	959,162.0	1,569,372.6	2,205,193.8	2,957,950.8
Cost of Rebar(Birr)	253,931.0	1,429,012.2	4,063,437.7	6,116,897.0	9,450,577.0
Total Cost (Birr)	776,212.7	2,388,174.2	5,632,810.3	8,322,090.8	12,408,527.8

Table 3-4 Bill of Quantities for HSC columns

	HSC (C95)				
	Shear Walls				
	G+5	*G+10*	*G+15*	*G+20*	*G+25*
Conc. (m^3)	613.3	1,373.2	2,162.9	2,928.4	3,594.0
Unit Price of Conc.(Birr/m^3)	4,936.5	4,936.5	4,936.5	4,936.5	4,936.5
Rebar (kg)					
dia 6mm					
dia 8mm	8,369.1	23,738.0	31,619.0	45,476.1	53,357.0
dia 12mm	6,147.8	16,081.8	21,937.2	32,163.6	37,018.9
dia16mm	18,826.4	47,717.4	67,952.6	94,434.9	114,670.1
dia 20mm	20,339.2	47,886.4	68,198.4	94,772.8	115,084.8
Unit Price of Rebar.(Birr/kg)					
dia 6mm					
dia 8mm	42.1	42.1	42.1	42.1	42.1
dia 12mm	39.1	39.1	39.1	39.1	39.1
dia16mm	39.1	39.1	39.1	39.1	39.1
dia 20mm	39.1	39.1	39.1	39.1	39.1
Cost of Conc.(Birr)	3,027,555.5	6,778,851.2	10,677,007.8	14,456,145.3	17,741,632.9
Cost of Rebar(Birr)	2,126,156.7	5,371,459.1	7,519,678.6	10,580,378.1	12,689,457.7
Total Cost (Birr)	5,153,712.1	12,150,310.2	18,196,686.4	25,036,523.4	30,431,090.6

Table 3-5 Bill of Quantities for HSC shear walls

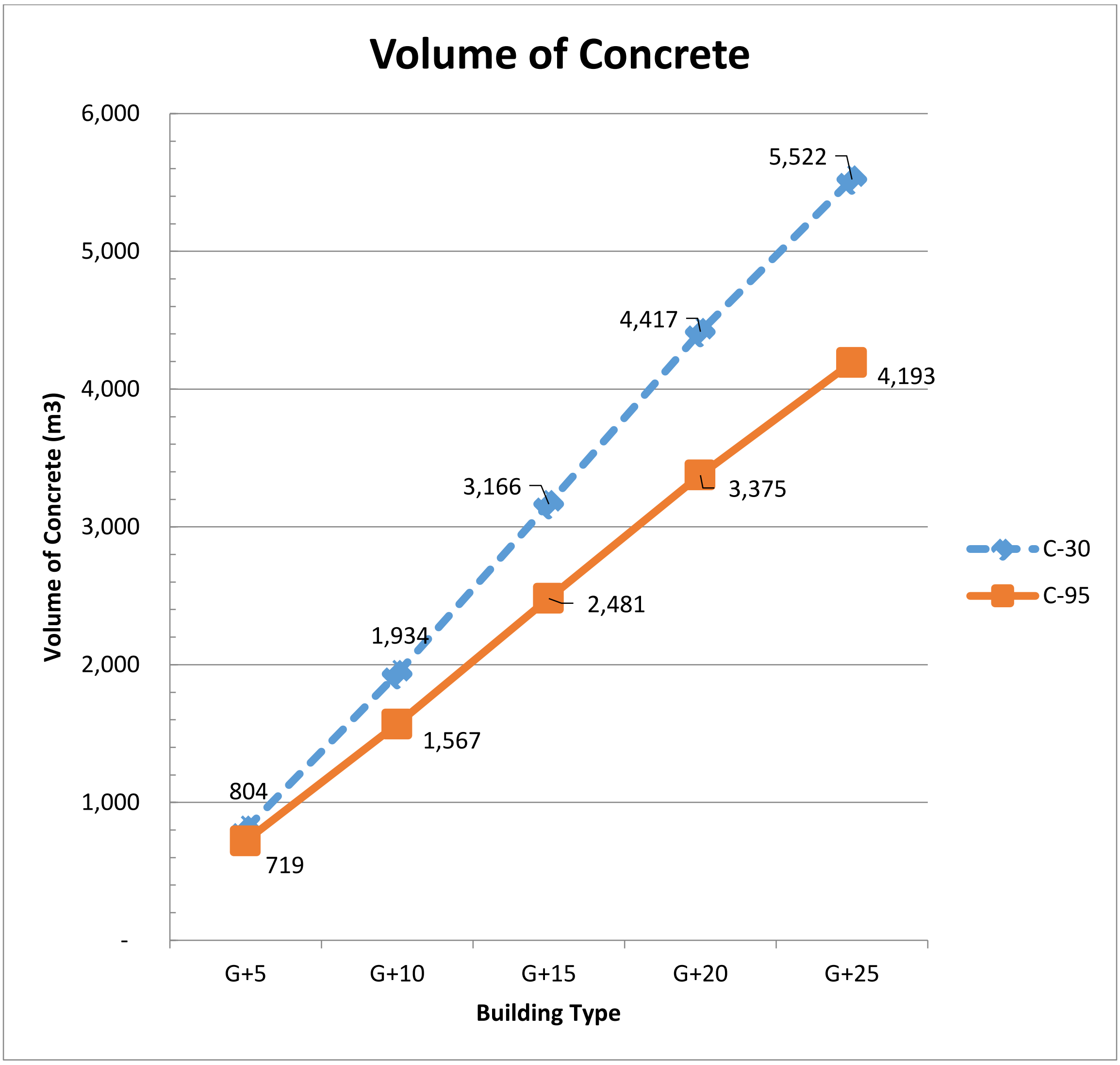

Figure 3-3 Total volume of concrete (m^3) used in Columns and Shear walls for NSC (C30) and HSC (C95)

	Volume of Concrete (m^3)			
Building Type	*NSC(C30)*	*HSC(C95)*	*Difference*	*%*
G+5 (H=15m)	804	719	85	11
G+10 (H=30m)	1,934	1,567	367	19
G+15 (H=45m)	3,166	2,481	685	22
G+20 (H=60m)	4,417	3,375	1,042	24
G+25 (H=75m)	5,522	4,193	1,329	24

Table 3-6 Total volume of concrete used in columns and shear walls (m^3)

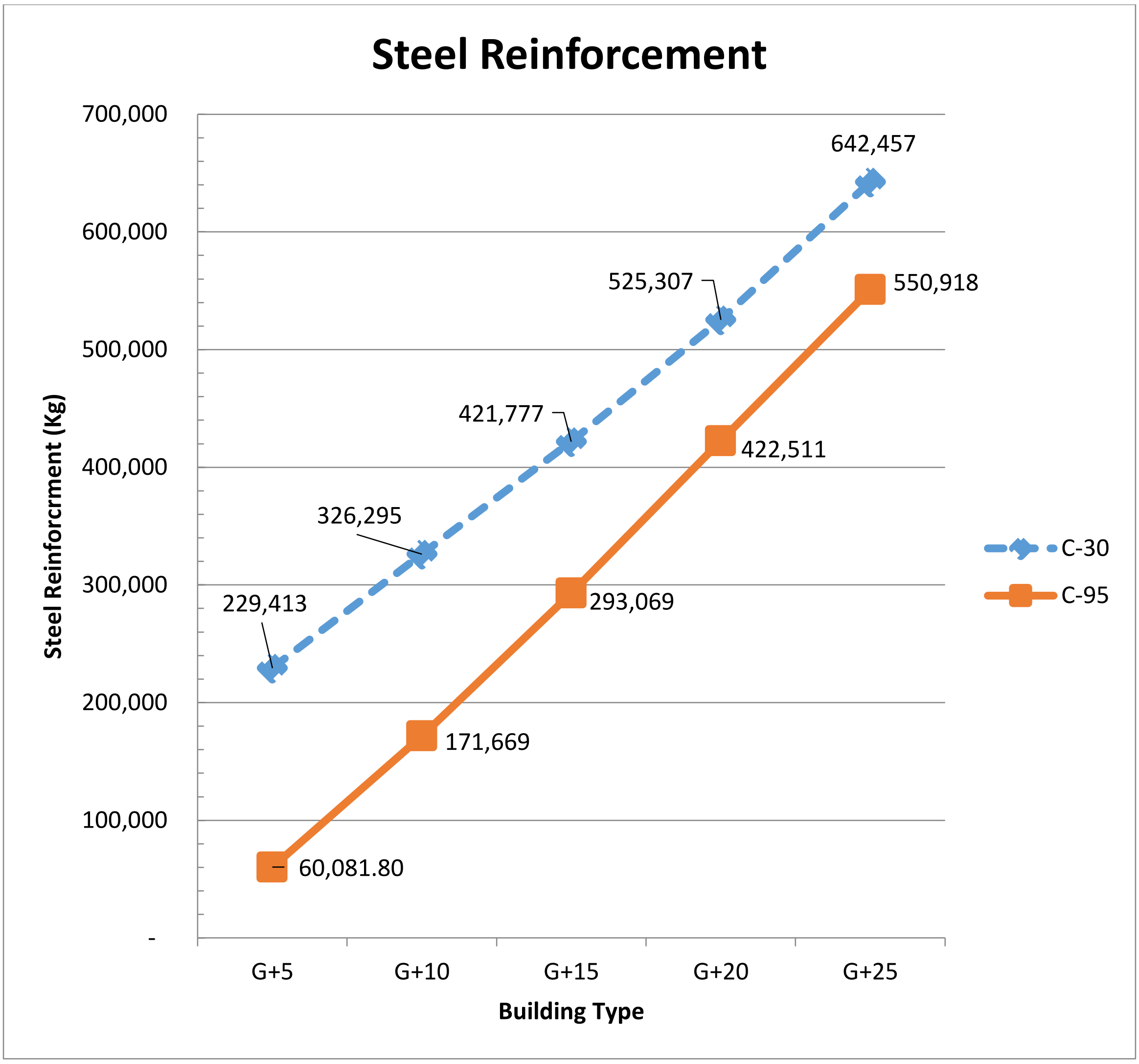

Figure 3-4 Total amount of reinforcement (Kg) used in columns and shear walls for NSC (C30) and HSC (C95)

	Steel Reinforcement (Kg)			
Building Type	*NSC(C30)*	*HSC(C95)*	*Difference*	*%*
G+5 (H=15m)	229,413	60,082	169,331	74
G+10 (H=30m)	366,295	171,669	154,626	47
G+15 (H=45m)	421,777	293,069	128,708	31
G+20 (H=60m)	525,307	422,511	102,795	20
G+25 (H=75m)	642,457	550,918	91,538	14

Table 3-7 Total amount of steel reinforcement used in columns and shear walls (Kg)

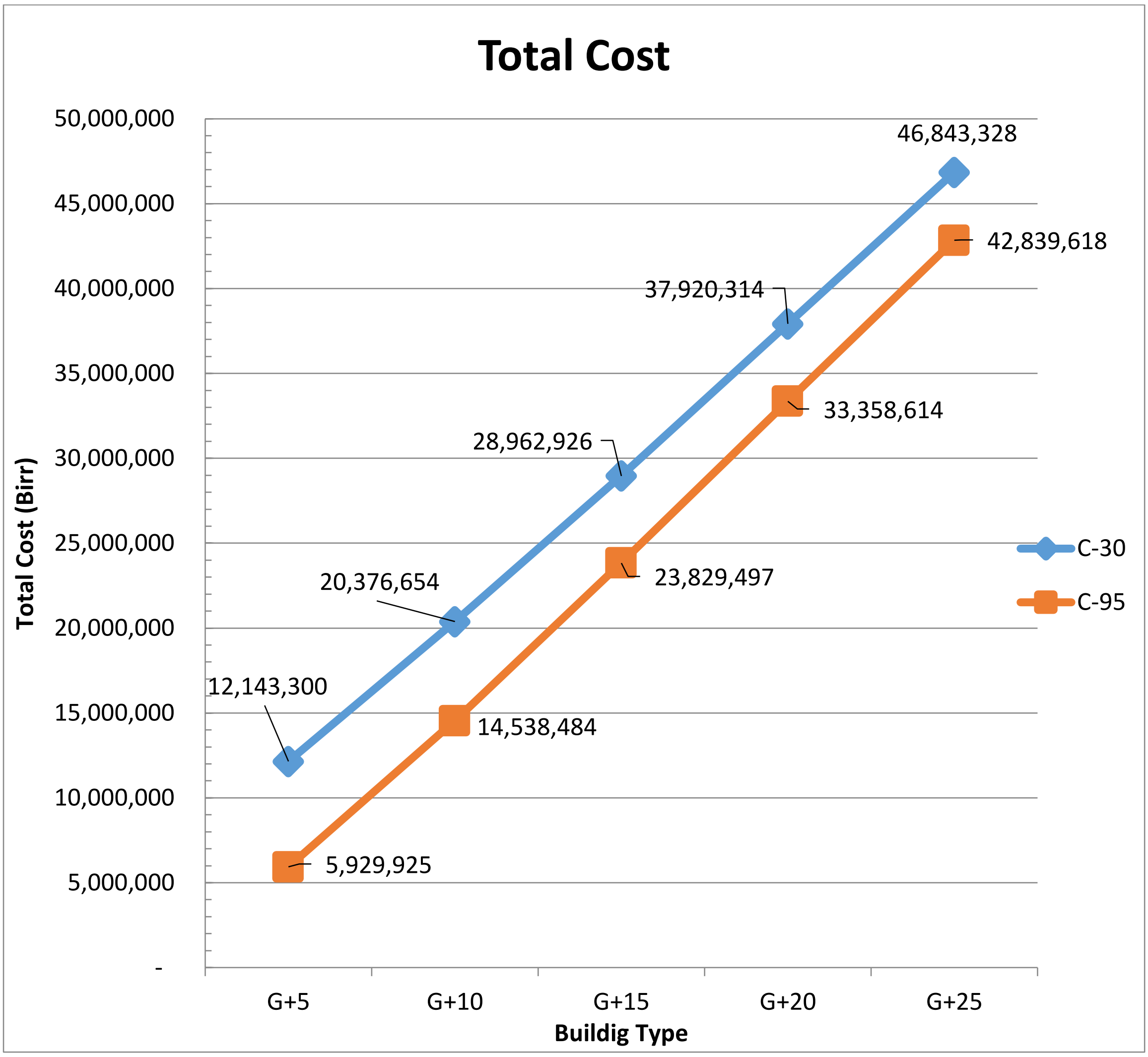

Figure 3-5 Total cost of concrete and reinforcement (Birr) used in columns and shear walls for NSC (C30) and HSC (C95)

	Cost (Birr)			
Building Type	*NSC(C30)*	*HSC(C95)*	*Difference*	*%*
G+5 (H=15m)	12,143,300	5,929,925	6,213,375	51
G+10 (H=30m)	20,376,654	14,538,484	5,838,170	29
G+15 (H=45m)	28,962,9262	23,829,497	5,133,429	18
G+20 (H=60m)	37,920,313	33,358,614	4,561,700	12
G+25 (H=75m)	46,843,328	42,839,618	4,003,709	9

Table 3-8 Total cost of concrete and reinforcement (Birr) used in columns and shear walls

CHAPTER 4—CONCLUSIONS

Although HSC is often considered as relatively new material, it is becoming accepted in most parts of the world. At the same time, material producers are responding to the demands for the material and are learning production techniques. However, further work is needed to fully use the advantages of HSC and to affirm its capabilities.

All materials for use in HSC must be carefully selected using all available techniques to insure uniform success. Mix proportions for HSC generally have been based on achieving a required compressive strength at a specified age. All materials must be optimized in concrete mix proportioning to achieve maximum strength.

Required strength, specified age, material characteristics, and type of application have strongly influenced the mix design. HSC mix proportioning has been found to be a more critical process than the proportioning of NSC mixes. In the area of structural analysis and design, it has been found that axially loaded columns with HSC can be designed in the same way as NSC columns.

The economic benefits of HSC are just now becoming fully apparent. Certainly as the use of HSC increases, additional and possibly even greater benefits will be realized. The above eight buildings have led the way in the use of HSC and have clearly demonstrated its economic advantages.

From the analysis and design output, as shown below in Table 3-1, it has been observed that the area saved while using HSC columns and shear walls is more significant as the height of the building increases from G+5 (H=15m) to G+25 (H=75m).

In addition, as shown below in Table 4-1 the following decrease in percentages were also perceived while using HSC columns and shear walls in G+5, G+10, G+15, G+20 and G+25 buildings.

Building Type	*% decrease in concrete*	*%decrease in reinforcement*	*% decrease in cost*
G+5 (H=15m)	11	74	51
G+10 (H=30m)	19	47	29
G+15(H=45m)	22	31	18
G+20(H=60m)	24	20	12
G+25(H=75m)	24	14	9

Table 4-1 Percentage decrease in concrete, reinforcement and cost when using HSC columns and shear walls

From the percentage decreases, it can be conclude that, as the height of building increases from G+5 to G+25, the % decrease in concrete is higher, % decrease in reinforcement is lower and the total % decrease in cost is lower. Thus the higher the building, the amount of area and concrete saved is significant. On the other hand, the amount of reinforcement saved decreases. So, it can be concluded that the overall cost of columns and shear walls saved is much significant for high-rise buildings lower than G+5. On the other hand, the higher the building, the more area is saved thus by allowing the user to have more usable floor area leading to an economical use of land.

CHAPTER 5—RECOMMENDATIONS

According to the conclusions, in order to save a more usable area in high-rise buildings, it is recommended to use HSC to NSC in columns and shear walls of high-rise buildings.

REFRENCED STANDARDS AND REPORTS

1. American Concrete Institute, (1997), "*State-of-the-Art Report on High-Strength Concrete,*" Journal of Structural Engineering Report No.ACI 363R-92*, USA*.

2. Arthur H.Nilson, (2010), *"Design of Concrete Structures",* 14th ed. Mc. Graw Hill Publishing, New York, USA.

3. Building and Construction Authority sustainable construction (2008) *"Design Guide of High Strength concrete to Singapore Standard CP65",* Building and Construction Authority, Singapore

4. Charles G. Salmon, John E. Johnson and Faris A. Malhas. (2009), "*Steel Structures Design and Behavior*," 5th ed. Pearson Prentice Hall, London.

5. Bekele Mekonnen, Assefa Desta, Negussie Tebedge, (1995), Ethiopian Building Code Standard - 1 *"Basis of Design and Actions on Structures",* Ministry of Works & Urban Development, Addis Ababa, Ethiopia

6. Negussie Tebedge, Asrat Tessema, Bekele Mekonnen, Mikyas Abayneh and Shifferaw Taye, (1995), Ethiopian Building Code Standard - 2, *"Structural*

use of Concrete", Ministry of Works & Urban Development, Addis Ababa, Ethiopia

7. Asrat Tessema, Bekele Mekonnen, Shifferaw Taye and Tesfaye Fanta, (1995), Ethiopian Building Code Standard - 8:- *"Design of Structures for Earthquake Resistance",* Ministry of Works & Urban Development, Addis Ababa, Ethiopia

8. Eurocode 2, (1992), "*Design of Concrete Structures*", European Committee for Standardization, Brussels, Belgium

9. Eurocode 8, (1998), "*Design of Structures for Earthquake Resistance*", European Committee for Standardization, Brussels, Belgium

10. James K.Wight, James G.MacGregor (2009), "*Reinforced Concrete Mechanics and Design*", Pearson Hall Publishing, New Jersey, USA

11. RF. Warner (1999), *"Concrete Structures",* Wesely Longman publishing, Melbourne, Australia.

APPENDIX A

Analysis input and output data for NSC

A-1 ETABS input data for NSC

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M A T E R I A L   P R O P E R T Y   D A T A

MATERIAL NAME	MATERIAL TYPE	DESIGN TYPE	MATERIAL DIR/PLANE	MODULUS OF ELASTICITY	POISSON'S RATIO	THERMAL COEFF	SHEAR MODULUS
STEEL	Iso	Steel	All	199.948	0.3000	1.1700E-05	76.903
CONC30	Iso	Concrete	All	31.000	0.2000	1.0000E-05	12.917
OTHER	Iso	None	All	199.948	0.3000	1.1700E-05	76.903
DUMMY	Iso	Concrete	All	31.000	0.2000	1.0000E-05	12.917

M A T E R I A L   P R O P E R T Y   M A S S   A N D   W E I G H T

MATERIAL NAME	MASS PER UNIT VOL	WEIGHT PER UNIT VOL
STEEL	7.8271E-12	7.6820E-08
CONC30	2.5000E-12	2.5000E-08
OTHER	7.8271E-12	7.6820E-08
DUMMY	0.0000E+00	0.0000E+00

M A T E R I A L   D E S I G N   D A T A   F O R   S T E E L   M A T E R I A L S

MATERIAL NAME	STEEL FY	STEEL FU	STEEL COST ($)
STEEL	0.345	0.448	271447.16

M A T E R I A L   D E S I G N   D A T A   F O R   C O N C R E T E   M A T E R I A L S

MATERIAL NAME	LIGHTWEIGHT CONCRETE	CONCRETE FCK	REBAR FYK	REBAR FYWK	LIGHTWT REDUC FACT
CONC30	No	0.025	0.400	0.400	N/A
DUMMY	No	0.025	0.400	0.400	N/A

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F R A M E   S E C T I O N   P R O P E R T Y   D A T A

FRAME SECTION NAME	MATERIAL NAME	SECTION SHAPE NAME OR NAME IN SECTION DATABASE FILE	CONC COL	CONC BEAM
GB25X40	CONC30	Rectangular		Yes
COL80X80	CONC30	Rectangular	Yes	
COL70X70	CONC30	Rectangular	Yes	
COL60X60	CONC30	Rectangular	Yes	
COL50X50	CONC30	Rectangular	Yes	
COL40X40	CONC30	Rectangular	Yes	
COL30X30	CONC30	Rectangular	Yes	
FB25X35	CONC30	Rectangular		Yes
COL90X90	CONC30	Rectangular	Yes	
FB25X50	CONC30	Rectangular		Yes
FB25X40	CONC30	Rectangular		Yes
COL100X100	CONC30	Rectangular	Yes	
FB25X60	CONC30	Rectangular		Yes
FB25X70	CONC30	Rectangular		Yes
FB25X80	CONC30	Rectangular		Yes
COL110X110	CONC30	Rectangular	Yes	
COL120X120	CONC30	Rectangular	Yes	
COL130X130	CONC30	Rectangular	Yes	
FB25X90	CONC30	Rectangular		Yes
FB30X90	CONC30	Rectangular		Yes
FB30X100	CONC30	Rectangular		Yes
COL140X140	CONC30	Rectangular	Yes	
FB40X110	CONC30	Rectangular		Yes
FB30X110	CONC30	Rectangular		Yes
FB40X120	CONC30	Rectangular		Yes
FB60X120	CONC30	Rectangular		Yes
FB70X120	CONC30	Rectangular		Yes
FB80X120	CONC30	Rectangular		Yes
FB90X120	CONC30	Rectangular		Yes

FB100X130	CONC30	Rectangular	Yes
FB100X140	CONC30	Rectangular	Yes
FB100X150	CONC30	Rectangular	Yes
FB110X140	CONC30	Rectangular	Yes
FB110X150	CONC30	Rectangular	Yes
FB120X150	CONC30	Rectangular	Yes
FB130X160	CONC30	Rectangular	Yes
FB140X170	CONC30	Rectangular	Yes
FB150X170	CONC30	Rectangular	Yes
FB150X180	CONC30	Rectangular	Yes

F R A M E   S E C T I O N   P R O P E R T Y   D A T A

FRAME SECTION NAME	SECTION DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOT	FLANGE THICK BOT
GB25X40	400.0000	250.0000	0.0000	0.0000	0.0000	0.0000
COL80X80	800.0000	800.0000	0.0000	0.0000	0.0000	0.0000
COL70X70	700.0000	700.0000	0.0000	0.0000	0.0000	0.0000
COL60X60	600.0000	600.0000	0.0000	0.0000	0.0000	0.0000
COL50X50	500.0000	500.0000	0.0000	0.0000	0.0000	0.0000
COL40X40	400.0000	400.0000	0.0000	0.0000	0.0000	0.0000
COL30X30	300.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB25X35	350.0000	250.0000	0.0000	0.0000	0.0000	0.0000
COL90X90	900.0000	900.0000	0.0000	0.0000	0.0000	0.0000
FB25X50	500.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB25X40	400.0000	250.0000	0.0000	0.0000	0.0000	0.0000
COL100X100	1000.0000	1000.0000	0.0000	0.0000	0.0000	0.0000
FB25X60	600.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB25X70	700.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB25X80	800.0000	250.0000	0.0000	0.0000	0.0000	0.0000
COL110X110	1100.0000	1100.0000	0.0000	0.0000	0.0000	0.0000
COL120X120	1200.0000	1200.0000	0.0000	0.0000	0.0000	0.0000
COL130X130	1300.0000	1300.0000	0.0000	0.0000	0.0000	0.0000
FB25X90	900.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB30X90	900.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB30X100	1000.0000	300.0000	0.0000	0.0000	0.0000	0.0000
COL140X140	1400.0000	1400.0000	0.0000	0.0000	0.0000	0.0000
FB40X110	1100.0000	400.0000	0.0000	0.0000	0.0000	0.0000
FB30X110	1100.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB40X120	1200.0000	400.0000	0.0000	0.0000	0.0000	0.0000
FB60X120	1200.0000	600.0000	0.0000	0.0000	0.0000	0.0000
FB70X120	1200.0000	700.0000	0.0000	0.0000	0.0000	0.0000
FB80X120	1200.0000	800.0000	0.0000	0.0000	0.0000	0.0000
FB90X120	1200.0000	900.0000	0.0000	0.0000	0.0000	0.0000
FB100X130	1300.0000	1000.0000	0.0000	0.0000	0.0000	0.0000
FB100X140	1400.0000	1000.0000	0.0000	0.0000	0.0000	0.0000
FB100X150	1500.0000	1000.0000	0.0000	0.0000	0.0000	0.0000
FB110X140	1400.0000	1100.0000	0.0000	0.0000	0.0000	0.0000
FB110X150	1500.0000	1100.0000	0.0000	0.0000	0.0000	0.0000
FB120X150	1200.0000	1500.0000	0.0000	0.0000	0.0000	0.0000
FB130X160	1600.0000	1300.0000	0.0000	0.0000	0.0000	0.0000
FB140X170	1700.0000	1400.0000	0.0000	0.0000	0.0000	0.0000
FB150X170	1700.0000	1500.0000	0.0000	0.0000	0.0000	0.0000
FB150X180	1800.0000	1500.0000	0.0000	0.0000	0.0000	0.0000

F R A M E   S E C T I O N   W E I G H T S   A N D   M A S S E S

FRAME SECTION NAME	TOTAL WEIGHT	TOTAL MASS
GB25X40	1371.0000	0.1371
COL80X80	4176.0000	0.4176
COL70X70	5218.5000	0.5219
COL60X60	4077.0000	0.4077
COL50X50	2550.0000	0.2550
COL40X40	1872.0000	0.1872
COL30X30	1404.0000	0.1404
FB25X35	14406.0000	1.4406
COL90X90	6135.7500	0.6136
FB25X50	3240.6250	0.3241
FB25X40	1176.0000	0.1176
COL100X100	3900.0000	0.3900
FB25X60	421.6875	0.0422
FB25X70	528.5000	0.0529
FB25X80	28.0000	0.0028
COL110X110	1996.5000	0.1997
COL120X120	2592.0000	0.2592
COL130X130	1394.2500	0.1394
FB25X90	27.8438	0.0028
FB30X90	426.2625	0.0426
FB30X100	451.5000	0.0452
COL140X140	294.0000	0.0294
FB40X110	18035.6000	1.8036
FB30X110	443.8500	0.0444
FB40X120	69.0000	0.0069
FB60X120	13751.1000	1.3751
FB70X120	3094.3500	0.3094
FB80X120	2719.2000	0.2719
FB90X120	17917.2000	1.7917
FB100X130	4566.2500	0.4566

FB100X140	3753.7500	0.3754
FB100X150	421.8750	0.0422
FB110X140	3114.6500	0.3115
FB110X150	2536.8750	0.2537
FB120X150	21260.2500	2.1260
FB130X160	6939.4000	0.6939
FB140X170	3915.1000	0.3915
FB150X170	6206.0625	0.6206
FB150X180	4816.1250	0.4816

C O N C R E T E   C O L U M N   D A T A

FRAME SECTION NAME	REINF CONFIGURATION LONGIT	LATERAL	REINF SIZE/TYPE	NUM BARS 3DIR/2DIR	NUM BARS CIRCULAR	BAR COVER
COL80X80	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL70X70	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL60X60	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL50X50	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL40X40	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL30X30	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL90X90	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL100X100	Rectangular	Ties	16Ø/Design	4/4	N/A	25.0000
COL110X110	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL120X120	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL130X130	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL140X140	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000

C O N C R E T E   B E A M   D A T A

FRAME SECTION NAME	TOP COVER	BOT COVER	TOP LEFT AREA	TOP RIGHT AREA	BOT LEFT AREA	BOT RIGHT AREA
GB25X40	50.0000	50.0000	0.000	0.000	0.000	0.000
FB25X35	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X50	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X40	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X60	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X70	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X80	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X90	25.0000	25.0000	0.000	0.000	0.000	0.000
FB30X90	25.0000	25.0000	0.000	0.000	0.000	0.000
FB30X100	25.0000	25.0000	0.000	0.000	0.000	0.000
FB40X110	25.0000	25.0000	0.000	0.000	0.000	0.000
FB30X110	25.0000	25.0000	0.000	0.000	0.000	0.000
FB40X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB60X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB70X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB80X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB90X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB100X130	25.0000	25.0000	0.000	0.000	0.000	0.000
FB100X140	25.0000	25.0000	0.000	0.000	0.000	0.000
FB100X150	25.0000	25.0000	0.000	0.000	0.000	0.000
FB110X140	25.0000	25.0000	0.000	0.000	0.000	0.000
FB110X150	25.0000	25.0000	0.000	0.000	0.000	0.000
FB120X150	25.0000	25.0000	0.000	0.000	0.000	0.000
FB130X160	25.0000	25.0000	0.000	0.000	0.000	0.000
FB140X170	25.0000	25.0000	0.000	0.000	0.000	0.000
FB150X170	25.0000	25.0000	0.000	0.000	0.000	0.000
FB150X180	25.0000	25.0000	0.000	0.000	0.000	0.000

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S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
DLTRANS	DEAD	N/A	1.0000		
LIVE	LIVE	N/A	0.0000		
EQXP	QUAKE	USER_LOADS	0.0000		
EQYP	QUAKE	USER_LOADS	0.0000		
EQXN	QUAKE	USER_LOADS	0.0000		
EQYN	QUAKE	USER_LOADS	0.0000		
WINDY	WIND	EUROCODE1 20	0.0000		
WINDX	WIND	EUROCODE1 20	0.0000		

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L O A D I N G   C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
COMB1	ADD	DLTRANS	Static	1.0000
COMB2	ADD	COMB1	Combo	0.7500
		EQXP	Static	1.0000
COMB3	ADD	COMB1	Combo	0.7500
		EQXP	Static	-1.0000
COMB4	ADD	COMB1	Combo	0.7500

		EQYP	Static	1.0000
COMB5	ADD	COMB1	Combo	0.7500
		EQYP	Static	-1.0000
COMB6	ADD	EQXN	Static	-1.0000
		COMB1	Combo	0.7500
COMB7	ADD	COMB1	Combo	0.7500
		EQXN	Static	1.0000
COMB8	ADD	COMB1	Combo	0.7500
		EQYN	Static	1.0000
COMB9	ADD	COMB1	Combo	0.7500
		EQYN	Static	-1.0000
COMB10	ADD	DLTRANS	Static	1.0000
		LIVE	Static	1.0000
COMB11	ADD	COMB1	Combo	0.7000
		WINDX	Static	1.0000
COMB12	ADD	COMB1	Combo	0.7000
		WINDY	Static	1.0000
ENVEQ	ENVE	COMB2	Combo	1.0000
		COMB3	Combo	1.0000
		COMB4	Combo	1.0000
		COMB5	Combo	1.0000
		COMB6	Combo	1.0000
		COMB7	Combo	1.0000
		COMB8	Combo	1.0000
		COMB9	Combo	1.0000

A U T O   S E I S M I C   U S E R   L O A D S
Case: EQXP, EQYP, EQXN, EQYN

AUTO SEISMIC INPUT DATA

Additional Eccentricity = 5%

SPECIFIED AUTO SEISMIC LOADS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	FX	FY	MZ
TWENTYFIFTH	D26	0.00	0.00	0.000
TWENTYFOURT	D25	0.00	0.00	0.000
TWENTYTHIRD	D25	0.00	0.00	0.000
TWENTYSECON	D23	0.00	0.00	0.000
TWENTYFIRST	D22	0.00	0.00	0.000
TWENTYTH	D21	0.00	0.00	0.000
NINTEENTH	D20	0.00	0.00	0.000
EEIGHTEENTH	D19	0.00	0.00	0.000
SEVENTEENTH	D18	0.00	0.00	0.000
SIXTEENTH	D17	0.00	0.00	0.000
FIFTEENTH	D16	0.00	0.00	0.000
FOURTEENTH	D15	0.00	0.00	0.000
THIRTEENTH	D14	0.00	0.00	0.000
TWELVETH	D13	0.00	0.00	0.000
ELEVENTH	D12	0.00	0.00	0.000
TENTH	D11	0.00	0.00	0.000
NINTH	D10	0.00	0.00	0.000
EIGTH	D9	0.00	0.00	0.000
SEVENTH	D8	0.00	0.00	0.000
SIXTH	D7	0.00	0.00	0.000
FIFTH	D6	0.00	0.00	0.000
FOURTH	D5	0.00	0.00	0.000
THIRD	D4	0.00	0.00	0.000
SECOND	D3	0.00	0.00	0.000
FIRST	D2	0.00	0.00	0.000
GROUND	D1	0.00	0.00	0.000

A-2 Sample Column Output for NSC

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C O N C R E T E   C O L U M N   D E S I G N   O U T P U T (EUROCODE 2-1992)

BIAXIAL P-M INTERACTION AND SHEAR DESIGN OF COLUMN-TYPE ELEMENTS

STORY	COLUMN	SECTION	STATION	<-------------------	REQUIRED REINFORCING	-------------------			-------------------->
ID	LINE	ID	ID	LONGITUDINAL	COMBO	SHEAR22	COMBO	SHEAR33	COMBO
TWENTYFIFTH	C66	COL30X30	0.000	1769.143	COMB10	0.061	COMB10	0.000	ENVEQ
TWENTYFIFTH	C66	COL30X30	1325.000	793.992	COMB10	0.138	COMB10	0.000	ENVEQ
TWENTYFIFTH	C66	COL30X30	2650.000	2307.589	COMB10	0.041	COMB10	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	0.000	1356.335	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	2650.000	759.630	COMB10	0.000	ENVEQ	0.000	ENVEQ

TWENTYTHIRD	C66	COL30X30	0.000	1224.297	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C66	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C66	COL30X30	2650.000	616.012	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	0.000	1333.145	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	1325.000	305.741	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	2650.000	687.531	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	0.000	1511.730	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	1325.000	527.289	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	2650.000	1078.260	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	0.000	2022.108	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	1325.000	1208.222	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	2650.000	1586.423	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	0.000	2575.035	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	1325.000	1879.233	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	2650.000	2214.316	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	0.000	3207.265	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	1325.000	2572.954	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	2650.000	2822.786	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	0.000	3808.330	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	1325.000	3328.236	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	2650.000	3539.955	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	0.000	4470.485	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	1325.000	3984.147	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	2650.000	4198.132	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	0.000	5235.379	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	1325.000	4822.603	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	2650.000	4975.633	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL40X40	0.000	2033.740	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL40X40	1325.000	1886.332	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL40X40	2650.000	1986.432	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL40X40	0.000	2935.837	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL40X40	1325.000	2701.786	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL40X40	2650.000	2838.775	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL40X40	0.000	3627.907	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL40X40	1325.000	3419.311	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL40X40	2650.000	3502.160	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL40X40	0.000	4339.763	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL40X40	1325.000	4150.267	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL40X40	2650.000	4240.222	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL40X40	0.000	5071.297	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL40X40	1325.000	4818.836	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL40X40	2650.000	4923.170	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL40X40	0.000	5690.154	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL40X40	1325.000	5507.264	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL40X40	2650.000	5614.267	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL50X50	0.000	2772.996	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL50X50	1325.000	2532.759	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL50X50	2650.000	2654.413	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C66	COL50X50	0.000	3553.060	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C66	COL50X50	1325.000	3361.938	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C66	COL50X50	2650.000	3407.333	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL50X50	0.000	4265.145	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL50X50	1325.000	4093.225	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL50X50	2650.000	4140.283	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL50X50	0.000	5030.088	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL50X50	1325.000	4820.935	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL50X50	2650.000	4868.599	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL50X50	0.000	5865.722	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL50X50	1325.000	5669.976	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL50X50	2650.000	5709.652	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL50X50	0.000	6558.488	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL50X50	1325.000	6425.315	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL50X50	2650.000	6468.434	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL50X50	0.000	7339.518	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL50X50	1325.000	7150.353	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL50X50	2650.000	7198.791	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL50X50	0.000	8088.135	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL50X50	1325.000	7948.356	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL50X50	2650.000	7974.850	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL80X80	0.000	2255.053	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL80X80	1300.000	2248.185	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL80X80	2600.000	2241.318	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	0.000	604.332	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	1325.000	542.556	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	2650.000	585.616	COMB10	0.000	ENVEQ	0.000	ENVEQ

TWENTYFOURTH	C67	COL30X30	0.000	1084.034	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C67	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C67	COL30X30	2650.000	809.358	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	0.000	1376.632	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	1325.000	500.172	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	2650.000	782.124	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	0.000	2536.316	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	1325.000	1774.389	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	2650.000	1973.555	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	0.000	3054.766	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	1325.000	2801.924	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	2650.000	3031.964	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	0.000	5131.102	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	1325.000	4539.865	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	2650.000	4983.771	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL40X40	0.000	2622.518	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL40X40	1325.000	2251.443	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL40X40	2650.000	2250.820	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL40X40	0.000	3997.778	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL40X40	1325.000	3596.270	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL40X40	2650.000	3587.630	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL40X40	0.000	4786.272	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL40X40	1325.000	4675.107	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL40X40	2650.000	4741.618	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL50X50	0.000	2637.561	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL50X50	1325.000	2186.708	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL50X50	2650.000	2495.327	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL50X50	0.000	3931.958	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL50X50	1325.000	3634.979	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL50X50	2650.000	3771.879	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL50X50	0.000	5292.765	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL50X50	1325.000	4936.573	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL50X50	2650.000	5130.968	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL50X50	0.000	6690.475	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL50X50	1325.000	6373.787	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL50X50	2650.000	6527.632	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL50X50	0.000	7896.079	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL50X50	1325.000	7596.649	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL50X50	2650.000	7801.210	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL60X60	0.000	4976.532	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL60X60	1325.000	4705.388	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL60X60	2650.000	4855.685	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL60X60	0.000	6263.090	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL60X60	1325.000	6054.213	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL60X60	2650.000	6105.859	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL60X60	0.000	7758.645	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL60X60	1325.000	7491.377	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL60X60	2650.000	7574.718	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C67	COL60X60	0.000	9201.452	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C67	COL60X60	1325.000	8968.543	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C67	COL60X60	2650.000	9039.744	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL60X60	0.000	10483.529	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL60X60	1325.000	10278.957	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL60X60	2650.000	10346.416	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL60X60	0.000	12030.669	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL60X60	1325.000	11746.318	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL60X60	2650.000	11837.387	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL60X60	0.000	13456.172	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL60X60	1325.000	13248.597	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL60X60	2650.000	13352.627	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL70X70	0.000	9557.918	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL70X70	1325.000	9332.813	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL70X70	2650.000	9389.325	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL70X70	0.000	11163.560	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL70X70	1325.000	10983.000	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL70X70	2650.000	11011.116	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL70X70	0.000	12695.000	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL70X70	1325.000	12488.882	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL70X70	2650.000	12519.922	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL70X70	0.000	13931.242	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL70X70	1325.000	13853.469	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL70X70	2650.000	13877.994	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL70X70	0.000	14324.241	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL70X70	1300.000	14275.564	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL70X70	2600.000	14230.158	COMB10	0.000	ENVEQ	0.000	ENVEQ

A-3 Foundation reactions for NSC

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L O A D I N G   C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
COMB1	ADD	DLTRANS	Static	1.0000

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S U P P O R T   R E A C T I O N S

STORY	POINT	LOAD	FX	FY	FZ	MX	MY	MZ
BASE	86	COMB1	5.47	-4.70	5229.05	4.413	5.195	0.001
BASE	87	COMB1	-0.07	1.21	9681.27	-1.212	-0.056	0.001
BASE	88	COMB1	-0.44	0.07	9810.07	-0.310	-0.364	0.004
BASE	89	COMB1	0.11	1.27	9710.60	-1.329	0.121	0.003
BASE	90	COMB1	0.03	1.27	9709.56	-1.340	0.048	0.003
BASE	91	COMB1	0.37	0.16	9817.36	-0.336	0.358	0.003
BASE	92	COMB1	-0.23	1.19	9607.79	-1.204	-0.216	0.001
BASE	93	COMB1	-5.65	-9.05	3234.86	8.497	-5.346	0.001
BASE	94	COMB1	-6.45	0.30	22244.79	-25.882	3.532	0.311
BASE	99	COMB1	5.21	-0.13	21504.79	-8.272	6.820	0.150
BASE	100	COMB1	-1.76	-0.31	30625.32	-10.345	5.718	0.175
BASE	105	COMB1	2.16	-0.03	30850.68	-13.899	6.671	0.175
BASE	106	COMB1	-6.04	0.25	22109.43	-25.882	10.943	0.311
BASE	113	COMB1	4.24	-0.14	20116.27	-5.897	2.933	0.107
BASE	114	COMB1	4.07	4.26	41058.05	-80.492	11.359	1.438
BASE	118	COMB1	-467.63	-451.77	4765.49	40.401	-45.257	0.065
BASE	119	COMB1	-7.05	-1.91	40521.70	-116.288	24.348	1.438
BASE	120	COMB1	0.06	-0.64	22813.24	-1.987	0.648	0.040
BASE	125	COMB1	-0.10	-0.61	22049.58	-2.804	0.547	0.040
BASE	126	COMB1	0.16	0.87	17865.50	-1.359	0.338	0.011
BASE	129	COMB1	-0.09	0.74	18473.40	-1.471	0.135	0.011
BASE	130	COMB1	1.16	10.94	36651.29	-101.270	31.905	1.438
BASE	133	COMB1	4.56	11.05	36349.92	-72.552	27.931	1.438
BASE	134	COMB1	0.13	-0.64	18800.17	-0.136	0.404	0.011
BASE	136	COMB1	-0.18	12.03	9013.11	-11.146	-0.094	0.003
BASE	137	COMB1	0.23	12.01	9463.59	-11.135	0.274	0.003
BASE	139	COMB1	-0.08	-0.59	18720.41	-0.391	0.233	0.011
BASE	140	COMB1	-0.07	0.51	22124.29	-2.889	0.952	0.032
BASE	145	COMB1	0.21	0.69	20339.25	-1.988	0.931	0.025
BASE	115	COMB1	469.42	-453.70	4788.14	40.579	45.456	-0.066
BASE	121	COMB1	2.15	435.14	2266.57	-32.944	1.668	-0.259
BASE	124	COMB1	-2.15	433.29	2258.43	-32.809	-1.666	0.259
BASE	144	COMB1	2.12	-459.40	2385.53	34.748	1.645	0.255
BASE	101	COMB1	499.02	479.88	5080.72	-42.964	48.338	0.068
BASE	141	COMB1	-2.13	-456.50	2372.05	34.529	-1.648	-0.256

BASE	104	COMB1	-496.04	477.30	5049.09	-42.739	-48.015	-0.068
BASE	131	COMB1	-437.31	-1.60	6376.06	0.099	-46.328	0.001
BASE	132	COMB1	437.90	-0.84	6363.91	0.037	46.401	0.000
BASE	127	COMB1	2.14	-410.87	2124.42	30.997	1.661	0.258
BASE	135	COMB1	2.12	413.10	2138.18	-31.193	1.650	-0.256
BASE	128	COMB1	-2.13	-411.74	2130.24	31.063	-1.647	-0.256
BASE	138	COMB1	-2.14	414.73	2149.21	-31.325	-1.659	0.258
BASE	303	COMB1	11.09	4.71	4884.01	-4.517	10.561	0.001
BASE	305	COMB1	-0.08	-1.10	9502.02	0.990	-0.041	0.001
BASE	314	COMB1	-0.41	-0.02	9802.93	-0.222	-0.219	0.004
BASE	307	COMB1	0.11	-1.22	9621.15	0.940	0.203	0.003
BASE	308	COMB1	0.04	-1.22	9616.61	0.930	0.137	0.003
BASE	315	COMB1	0.36	-0.16	9766.78	-0.044	0.425	0.003
BASE	310	COMB1	-0.20	-1.07	9433.74	0.945	-0.157	0.001
BASE	304	COMB1	-11.24	9.06	3043.72	-8.658	-10.614	0.001
BASE	24	COMB1	-0.22	-11.96	8770.88	10.741	-0.152	0.003
BASE	33	COMB1	0.23	-11.97	8809.76	10.733	0.261	0.003
BASE	337	COMB1	-269.19	5.84	5155.67	-5.356	-9.309	0.258
BASE	338	COMB1	-38.92	0.61	5225.24	-1.778	-1.210	-0.176
BASE	339	COMB1	1.87	3.93	5217.66	-3.573	0.733	0.013
BASE	340	COMB1	4.10	3.63	5203.96	-3.637	0.394	0.032
BASE	341	COMB1	-1.22	2.93	5200.90	-3.172	-0.071	0.000
BASE	342	COMB1	-6.43	3.63	5212.39	-3.632	-0.529	-0.032
BASE	343	COMB1	-3.84	3.93	5234.10	-3.566	-0.843	-0.013
BASE	344	COMB1	37.75	0.59	5248.77	-1.772	1.150	0.177
BASE	345	COMB1	270.18	5.87	5184.51	-5.381	9.364	-0.259
BASE	346	COMB1	-215.21	-0.01	4533.07	-0.003	-6.938	0.000
BASE	347	COMB1	-22.57	-0.01	4565.40	-0.003	-0.260	0.000
BASE	348	COMB1	9.94	-0.02	4539.92	0.008	1.267	0.000
BASE	349	COMB1	8.62	-0.02	4514.51	0.005	0.705	0.000
BASE	350	COMB1	0.84	-0.01	4504.60	-0.004	0.075	0.000
BASE	351	COMB1	-6.89	-0.01	4513.63	-0.005	-0.552	0.000
BASE	352	COMB1	-8.07	-0.01	4537.97	-0.004	-1.103	0.000
BASE	353	COMB1	24.66	0.00	4561.96	-0.007	0.442	0.000
BASE	354	COMB1	217.25	0.00	4527.58	-0.005	7.133	0.000
BASE	355	COMB1	-253.61	-5.54	4862.67	5.072	-8.759	-0.244
BASE	356	COMB1	-36.99	-0.69	4927.39	1.777	-1.205	0.168
BASE	357	COMB1	1.48	-3.93	4919.80	3.552	0.667	-0.012
BASE	358	COMB1	3.79	-3.63	4906.04	3.609	0.357	-0.032
BASE	359	COMB1	-1.22	-2.93	4902.07	3.148	-0.084	0.000
BASE	360	COMB1	-6.17	-3.63	4911.96	3.612	-0.521	0.032
BASE	361	COMB1	-3.69	-3.93	4931.48	3.554	-0.822	0.012
BASE	362	COMB1	35.32	-0.69	4944.32	1.778	1.081	-0.168
BASE	363	COMB1	253.54	-5.57	4883.57	5.094	8.720	0.245
BASE	364	COMB1	1.78	-249.65	3949.71	12.556	2.015	0.004
BASE	365	COMB1	-0.63	-43.40	3995.32	1.560	0.713	-0.102
BASE	366	COMB1	5.45	0.34	3974.80	-0.634	4.313	0.189

BASE	367	COMB1	5.46	-1.92	3982.17	0.703	4.315	-0.189
BASE	368	COMB1	-0.63	42.19	4009.96	-1.511	0.712	0.102
BASE	369	COMB1	1.77	249.99	3971.07	-12.593	2.005	-0.004
BASE	370	COMB1	5.91	-278.55	4124.69	15.836	4.759	-0.197
BASE	371	COMB1	-0.77	-24.15	4236.72	2.079	0.694	0.111
BASE	372	COMB1	1.77	249.02	4209.62	-12.648	2.008	-0.005
BASE	373	COMB1	1.76	-265.94	4435.52	13.613	1.985	0.004
BASE	374	COMB1	-0.85	21.95	4475.16	-1.940	0.695	-0.119
BASE	375	COMB1	6.28	292.36	4367.49	-16.629	5.061	0.209
BASE	376	COMB1	-1.77	-249.63	3960.44	12.526	-1.999	-0.004
BASE	377	COMB1	0.63	-42.63	4005.19	1.476	-0.706	0.102
BASE	378	COMB1	-5.45	0.86	3983.51	-0.703	-4.303	-0.189
BASE	379	COMB1	-5.45	-3.82	3994.21	0.875	-4.304	0.189
BASE	380	COMB1	0.63	40.42	4026.16	-1.352	-0.706	-0.102
BASE	381	COMB1	-1.78	249.84	3990.39	-12.548	-2.009	0.004
BASE	382	COMB1	-5.89	-277.83	4105.74	15.802	-4.735	0.197
BASE	383	COMB1	0.77	-24.84	4218.99	2.140	-0.689	-0.111
BASE	384	COMB1	-1.77	247.35	4193.80	-12.538	-2.004	0.005
BASE	385	COMB1	-1.76	-263.59	4410.43	13.456	-1.982	-0.004
BASE	386	COMB1	0.84	22.84	4448.76	-2.026	-0.687	0.118
BASE	387	COMB1	-6.25	291.47	4340.48	-16.606	-5.022	-0.208
BASE	122	COMB1	-0.35	10.20	11331.98	-9.108	-0.175	0.008
BASE	123	COMB1	0.36	9.87	11210.58	-8.862	0.420	0.008
BASE	142	COMB1	0.44	-11.84	10685.76	9.373	0.621	0.008
BASE	143	COMB1	-0.41	-11.87	10302.85	9.432	-0.097	0.008
BASE	111	COMB1	1.21	2.46	3136.98	-0.163	0.727	-0.001
BASE	112	COMB1	1.94	-290.91	1680.44	15.337	0.973	0.079
BASE	108	COMB1	1.96	295.49	1709.31	-15.598	0.988	-0.080
BASE	109	COMB1	1.23	-4.40	3166.29	0.221	0.743	0.001
BASE	96	COMB1	-1.26	-1.63	2850.48	-0.009	-0.760	0.001
BASE	95	COMB1	-2.00	-262.35	1512.39	13.802	-1.018	-0.082
BASE	107	COMB1	-1.98	269.49	1556.82	-14.207	-0.995	0.081
BASE	98	COMB1	-1.25	-1.43	2896.87	0.102	-0.748	-0.001
BASE	926	COMB1	-0.11	-7.94	3144.70	0.136	-0.280	0.015
BASE	927	COMB1	-2.40	-58.93	3132.50	1.224	-1.718	0.013
BASE	928	COMB1	-1.55	-213.06	3086.02	6.445	-0.673	-0.034
BASE	929	COMB1	-1.55	215.45	3135.17	-6.523	-0.662	0.034
BASE	930	COMB1	-2.41	58.33	3175.87	-1.200	-1.716	-0.013
BASE	931	COMB1	-0.10	6.24	3181.31	-0.084	-0.270	-0.015
BASE	932	COMB1	0.07	-11.62	2848.93	0.300	0.240	-0.015
BASE	933	COMB1	2.35	-56.70	2829.23	1.237	1.668	-0.013
BASE	934	COMB1	1.48	-193.92	2780.62	5.863	0.602	0.034
BASE	935	COMB1	1.54	197.82	2856.49	-5.992	0.656	-0.034
BASE	936	COMB1	2.40	55.97	2896.71	-1.207	1.715	0.013
BASE	937	COMB1	0.09	9.08	2906.40	-0.223	0.267	0.015
BASE	110	COMB1	0.99	-1.01	3124.95	0.030	0.562	0.000
BASE	1044	COMB1	-0.11	-7.20	3150.21	0.291	-0.161	0.017

BASE	1045	COMB1	-1.68	-4.97	3136.45	0.179	-1.129	0.000
BASE	1046	COMB1	-0.22	-2.21	3128.96	0.058	-0.268	-0.016
BASE	1047	COMB1	-0.22	0.19	3121.64	0.002	-0.268	0.016
BASE	1048	COMB1	-1.68	2.95	3121.81	-0.120	-1.132	0.000
BASE	1049	COMB1	-0.12	5.19	3128.22	-0.232	-0.171	-0.017
BASE	97	COMB1	-1.01	-1.74	2859.25	0.053	-0.572	0.000
BASE	1050	COMB1	0.10	-5.08	2884.30	0.210	0.157	-0.017
BASE	1051	COMB1	1.68	-3.86	2872.22	0.135	1.127	0.000
BASE	1052	COMB1	0.21	-2.04	2864.75	0.048	0.262	0.016
BASE	1053	COMB1	0.21	-1.42	2853.01	0.057	0.261	-0.016
BASE	1054	COMB1	1.67	0.45	2848.80	-0.031	1.126	0.000
BASE	1055	COMB1	0.10	1.78	2849.29	-0.110	0.153	0.017
Summation	0, 0, Base	COMB1	0.00	0.00	1053235.03	18418555.644	-17378010.36	25149.707

APPENDIX B

Analysis input and output data for HSC

B-1 ETABS input data for HSC

ETABS v9.7.4 File:HIGH STRENGTH C95 G+25 Units:KN-mm February 3, 2015 10:25 PAGE 2

M A T E R I A L   P R O P E R T Y   D A T A

MATERIAL NAME	MATERIAL TYPE	DESIGN TYPE	MATERIAL DIR/PLANE	MODULUS OF ELASTICITY	POISSON'S RATIO	THERMAL COEFF	SHEAR MODULUS
STEEL	Iso	Steel	All	199.948	0.3000	1.1700E-05	76.903
CONC95	Iso	Concrete	All	42.000	0.2000	1.0000E-05	17.500
CONC45	Iso	Concrete	All	34.000	0.2000	1.0000E-05	14.167
DUMMY	Iso	Concrete	All	34.000	0.2000	1.0000E-05	14.167

M A T E R I A L   P R O P E R T Y   M A S S   A N D   W E I G H T

MATERIAL NAME	MASS PER UNIT VOL	WEIGHT PER UNIT VOL
STEEL	7.8271E-12	7.6820E-08
CONC95	2.5000E-12	2.5000E-08
CONC45	2.5000E-12	2.5000E-08
DUMMY	0.0000E+00	0.0000E+00

M A T E R I A L   D E S I G N   D A T A   F O R   S T E E L   M A T E R I A L S

MATERIAL NAME	STEEL FY	STEEL FU	STEEL COST ($)
STEEL	0.345	0.448	271447.16

M A T E R I A L   D E S I G N   D A T A   F O R   C O N C R E T E   M A T E R I A L S

MATERIAL NAME	LIGHTWEIGHT CONCRETE	CONCRETE FCK	REBAR FYK	REBAR FYWK	LIGHTWT REDUC FACT
CONC95	No	0.080	0.400	0.400	N/A
CONC45	No	0.035	0.400	0.400	N/A
DUMMY	No	0.035	0.400	0.400	N/A

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F R A M E   S E C T I O N   P R O P E R T Y   D A T A

FRAME SECTION NAME	MATERIAL NAME	SECTION SHAPE NAME OR NAME IN SECTION DATABASE FILE	CONC COL	CONC BEAM
GB25X50	CONC45	Rectangular		Yes
COL80X80	CONC95	Rectangular	Yes	
COL70X70	CONC95	Rectangular	Yes	
COL60X60	CONC95	Rectangular	Yes	
COL50X50	CONC95	Rectangular	Yes	
COL40X40	CONC95	Rectangular	Yes	
COL30X30	CONC95	Rectangular	Yes	
FB30X60	CONC45	Rectangular		Yes
FB35X80	CONC45	Rectangular		Yes
FB25X35	CONC45	Rectangular		Yes
FB25X40	CONC45	Rectangular		Yes
FB30X90	CONC45	Rectangular		Yes
COL90X90	CONC95	Rectangular	Yes	
FB30X110	CONC45	Rectangular		Yes
FB35X120	CONC45	Rectangular		Yes
FB35X130	CONC45	Rectangular		Yes
FB35X140	CONC45	Rectangular		Yes
FB35X150	CONC45	Rectangular		Yes
FB45X160	CONC45	Rectangular		Yes
FB55X180	CONC45	Rectangular		Yes
FB65X180	CONC45	Rectangular		Yes
FB80X180	CONC45	Rectangular		Yes
FB100X200	CONC45	Rectangular		Yes

F R A M E   S E C T I O N   P R O P E R T Y   D A T A

FRAME SECTION NAME	SECTION DEPTH	FLANGE WIDTH TOP	FLANGE THICK TOP	WEB THICK	FLANGE WIDTH BOT	FLANGE THICK BOT
GB25X50	500.0000	250.0000	0.0000	0.0000	0.0000	0.0000
COL80X80	800.0000	800.0000	0.0000	0.0000	0.0000	0.0000
COL70X70	700.0000	700.0000	0.0000	0.0000	0.0000	0.0000
COL60X60	600.0000	600.0000	0.0000	0.0000	0.0000	0.0000
COL50X50	500.0000	500.0000	0.0000	0.0000	0.0000	0.0000

COL40X40	400.0000	400.0000	0.0000	0.0000	0.0000	0.0000
COL30X30	300.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB30X60	600.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB35X80	800.0000	350.0000	0.0000	0.0000	0.0000	0.0000
FB25X35	350.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB25X40	400.0000	250.0000	0.0000	0.0000	0.0000	0.0000
FB30X90	900.0000	300.0000	0.0000	0.0000	0.0000	0.0000
COL90X90	900.0000	900.0000	0.0000	0.0000	0.0000	0.0000
FB30X110	1100.0000	300.0000	0.0000	0.0000	0.0000	0.0000
FB35X120	1200.0000	350.0000	0.0000	0.0000	0.0000	0.0000
FB35X130	1300.0000	350.0000	0.0000	0.0000	0.0000	0.0000
FB35X140	1400.0000	350.0000	0.0000	0.0000	0.0000	0.0000
FB35X150	1500.0000	350.0000	0.0000	0.0000	0.0000	0.0000
FB45X160	1600.0000	450.0000	0.0000	0.0000	0.0000	0.0000
FB55X180	1800.0000	550.0000	0.0000	0.0000	0.0000	0.0000
FB65X180	1800.0000	650.0000	0.0000	0.0000	0.0000	0.0000
FB80X180	1800.0000	800.0000	0.0000	0.0000	0.0000	0.0000
FB100X200	2000.0000	1000.0000	0.0000	0.0000	0.0000	0.0000

F R A M E   S E C T I O N   W E I G H T S   A N D   M A S S E S

FRAME SECTION NAME	TOTAL WEIGHT	TOTAL MASS
GB25X50	1786.2500	0.1786
COL80X80	1488.0000	0.1488
COL70X70	1984.5000	0.1985
COL60X60	2403.0000	0.2403
COL50X50	3075.0000	0.3075
COL40X40	4224.0000	0.4224
COL30X30	2693.2500	0.2693
FB30X60	7296.5250	0.7297
FB35X80	7225.4000	0.7225
FB25X35	17532.7031	1.7533
FB25X40	685.8750	0.0686
FB30X90	982.1250	0.0982
COL90X90	182.2500	0.0182
FB30X110	2187.9000	0.2188
FB35X120	1399.6500	0.1400
FB35X130	2123.1438	0.2123
FB35X140	142.7125	0.0143
FB35X150	984.3750	0.0984
FB45X160	1254.6000	0.1255
FB55X180	9432.2250	0.9432
FB65X180	6413.0625	0.6413
FB80X180	7749.0000	0.7749
FB100X200	38615.0000	3.8615

C O N C R E T E   C O L U M N   D A T A

FRAME SECTION NAME	REINF CONFIGURATION LONGIT	LATERAL	REINF SIZE/TYPE	NUM BARS 3DIR/2DIR	NUM BARS CIRCULAR	BAR COVER
COL80X80	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL70X70	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL60X60	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL50X50	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL40X40	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL30X30	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000
COL90X90	Rectangular	Ties	16Ø/Design	3/3	N/A	25.0000

C O N C R E T E   B E A M   D A T A

FRAME SECTION NAME	TOP COVER	BOT COVER	TOP LEFT AREA	TOP RIGHT AREA	BOT LEFT AREA	BOT RIGHT AREA
GB25X50	50.0000	50.0000	0.000	0.000	0.000	0.000
FB30X60	25.0000	25.0000	0.000	0.000	0.000	0.000
FB35X80	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X35	25.0000	25.0000	0.000	0.000	0.000	0.000
FB25X40	25.0000	25.0000	0.000	0.000	0.000	0.000
FB30X90	25.0000	25.0000	0.000	0.000	0.000	0.000
FB30X110	25.0000	25.0000	0.000	0.000	0.000	0.000
FB35X120	25.0000	25.0000	0.000	0.000	0.000	0.000
FB35X130	25.0000	25.0000	0.000	0.000	0.000	0.000
FB35X140	25.0000	25.0000	0.000	0.000	0.000	0.000
FB35X150	25.0000	25.0000	0.000	0.000	0.000	0.000
FB45X160	25.0000	25.0000	0.000	0.000	0.000	0.000
FB55X180	25.0000	25.0000	0.000	0.000	0.000	0.000
FB65X180	25.0000	25.0000	0.000	0.000	0.000	0.000
FB80X180	25.0000	25.0000	0.000	0.000	0.000	0.000
FB100X200	25.0000	25.0000	0.000	0.000	0.000	0.000

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S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
DLTRANS	DEAD	N/A	1.0000		
LIVE	LIVE	N/A	0.0000		
EQXP	QUAKE	USER_LOADS	0.0000		
EQYP	QUAKE	USER_LOADS	0.0000		
EQXN	QUAKE	USER_LOADS	0.0000		
EQYN	QUAKE	USER_LOADS	0.0000		
WINDX	WIND	EUROCODE1 20	0.0000		
WINDY	WIND	EUROCODE1 20	0.0000		

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L O A D I N G   C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
COMB1	ADD	DLTRANS	Static	1.0000
COMB2	ADD	COMB1	Combo	0.7500
		EQXP	Static	1.0000
COMB3	ADD	COMB1	Combo	0.7500
		EQXP	Static	-1.0000
COMB4	ADD	COMB1	Combo	0.7500
		EQYP	Static	1.0000
COMB5	ADD	COMB1	Combo	0.7500
		EQYP	Static	-1.0000
COMB7	ADD	COMB1	Combo	0.7500
		EQXN	Static	1.0000
COMB8	ADD	COMB1	Combo	0.7500
		EQYN	Static	1.0000
COMB9	ADD	COMB1	Combo	0.7500
		EQYN	Static	-1.0000
COMB10	ADD	DLTRANS	Static	1.0000
		LIVE	Static	1.0000
COMB11	ADD	COMB1	Combo	0.7000
		WINDX	Static	1.0000
COMB12	ADD	COMB1	Combo	0.7000
		WINDY	Static	1.0000
COMB6	ADD	COMB1	Combo	0.7500
		EQXN	Static	-1.0000
ENVEQ	ENVE	COMB2	Combo	1.0000
		COMB3	Combo	1.0000
		COMB4	Combo	1.0000
		COMB5	Combo	1.0000
		COMB6	Combo	1.0000
		COMB7	Combo	1.0000
		COMB8	Combo	1.0000
		COMB9	Combo	1.0000

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A U T O   S E I S M I C   U S E R   L O A D S
Case: EQXP,EQXN,EQYP,EQYN

AUTO SEISMIC INPUT DATA

Additional Eccentricity = 5%

SPECIFIED AUTO SEISMIC LOADS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	FX	FY	MZ
TWENTYFIFTH	D25	0.00	0.00	0.000
TWENTYFOURT	D25	0.00	0.00	0.000
TWENTYTHIRD	D24	0.00	0.00	0.000
TWENTYSECON	D23	0.00	0.00	0.000
TWENTYFIRST	D22	0.00	0.00	0.000
TWENTYTH	D21	0.00	0.00	0.000
NINTEENTH	D20	0.00	0.00	0.000
EEIGHTEENTH	D19	0.00	0.00	0.000
SEVENTEENTH	D18	0.00	0.00	0.000
SIXTEENTH	D17	0.00	0.00	0.000
FIFTEENTH	D16	0.00	0.00	0.000
FOURTEENTH	D15	0.00	0.00	0.000
THIRTEENTH	D14	0.00	0.00	0.000
TWELVETH	D13	0.00	0.00	0.000
ELEVENTH	D12	0.00	0.00	0.000
TENTH	D11	0.00	0.00	0.000
NINTH	D9	0.00	0.00	0.000

NINTH	D10	0.00	0.00	0.000
EIGTH	D9	0.00	0.00	0.000
SEVENTH	D8	0.00	0.00	0.000
SIXTH	D7	0.00	0.00	0.000
FIFTH	D6	0.00	0.00	0.000
FOURTH	D5	0.00	0.00	0.000
THIRD	D4	0.00	0.00	0.000
SECOND	D3	0.00	0.00	0.000
FIRST	D2	0.00	0.00	0.000
GROUND	D1	0.00	0.00	0.000

B-2 Sample Column Output for HSC

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C O N C R E T E C O L U M N D E S I G N O U T P U T (EUROCODE 2-1992)

BIAXIAL P-M INTERACTION AND SHEAR DESIGN OF COLUMN-TYPE ELEMENTS

STORY	COLUMN	SECTION	STATION	<-------------------REQUIRED REINFORCING------------------->					
ID	LINE	ID	ID	LONGITUDINAL	COMBO	SHEAR22	COMBO	SHEAR33	COMBO
TWENTYFIFTH	C66	COL30X30	0.000	1280.132	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C66	COL30X30	1325.000	343.309	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C66	COL30X30	2650.000	1526.383	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	0.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C66	COL30X30	2650.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C66	COL30X30	0.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C66	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C66	COL30X30	2650.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	0.000	327.990	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	1325.000	326.704	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C66	COL30X30	2650.000	325.419	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	0.000	410.340	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	1325.000	409.054	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C66	COL30X30	2650.000	407.768	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	0.000	492.467	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	1325.000	491.182	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C66	COL30X30	2650.000	489.896	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	0.000	577.701	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	1325.000	575.416	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C66	COL30X30	2650.000	573.130	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	0.000	663.220	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	1325.000	660.935	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C66	COL30X30	2650.000	658.649	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	0.000	748.621	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	1325.000	746.335	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C66	COL30X30	2650.000	744.050	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	0.000	834.014	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	1325.000	831.729	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C66	COL30X30	2650.000	829.443	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	0.000	919.355	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	1325.000	917.070	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C66	COL30X30	2650.000	914.784	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL30X30	0.000	1004.678	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL30X30	1325.000	1002.393	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C66	COL30X30	2650.000	1000.107	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL30X30	0.000	1089.982	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL30X30	1325.000	1087.697	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C66	COL30X30	2650.000	1085.411	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL30X30	0.000	1175.272	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL30X30	1325.000	1172.987	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C66	COL30X30	2650.000	1170.701	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL30X30	0.000	1260.555	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL30X30	1325.000	1258.269	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C66	COL30X30	2650.000	1255.984	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL30X30	0.000	1345.812	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL30X30	1325.000	1343.526	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C66	COL30X30	2650.000	1341.241	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL30X30	0.000	1679.891	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL30X30	1325.000	1429.005	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C66	COL30X30	2650.000	1535.953	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL30X30	0.000	2452.997	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL30X30	1325.000	2195.798	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C66	COL30X30	2650.000	2305.873	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C66	COL30X30	0.000	3338.367	COMB10	0.000	ENVEQ	0.000	ENVEQ

SEVENTH	C66	COL30X30	1325.000	3037.485	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C66	COL30X30	2650.000	3141.279	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL30X30	0.000	3763.195	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL30X30	1325.000	3602.083	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C66	COL30X30	2650.000	3718.318	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL30X30	0.000	4980.631	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL30X30	1325.000	4626.070	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C66	COL30X30	2650.000	4865.826	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL30X30	0.000	5698.029	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL30X30	1325.000	5406.274	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C66	COL30X30	2650.000	5505.609	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL30X30	0.000	6505.159	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL30X30	1325.000	6234.380	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C66	COL30X30	2650.000	6375.484	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL40X40	0.000	2048.009	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL40X40	1325.000	2044.438	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C66	COL40X40	2650.000	2040.867	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL40X40	0.000	2137.812	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL40X40	1325.000	2134.241	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C66	COL40X40	2650.000	2130.669	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL40X40	0.000	2179.075	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL40X40	1250.000	2175.706	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C66	COL40X40	2500.000	2172.337	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	0.000	779.865	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	1325.000	270.000	ENVEQ	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIFTH	C67	COL30X30	2650.000	1034.096	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C67	COL30X30	0.000	301.676	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C67	COL30X30	1325.000	300.390	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFOURTH	C67	COL30X30	2650.000	299.105	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	0.000	452.837	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	1325.000	451.552	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTHIRD	C67	COL30X30	2650.000	450.266	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	0.000	604.377	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	1325.000	603.091	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYSECOND	C67	COL30X30	2650.000	601.806	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	0.000	756.435	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	1325.000	755.149	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYFIRST	C67	COL30X30	2650.000	753.863	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	0.000	909.107	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	1325.000	907.821	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWENTYTH	C67	COL30X30	2650.000	906.535	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL30X30	0.000	1064.925	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL30X30	1325.000	1062.639	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTEENTH	C67	COL30X30	2650.000	1060.354	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL30X30	0.000	1221.211	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL30X30	1325.000	1218.925	COMB10	0.000	ENVEQ	0.000	ENVEQ
EEIGHTEENTH	C67	COL30X30	2650.000	1216.639	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL30X30	0.000	1378.040	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL30X30	1325.000	1375.754	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTEENTH	C67	COL30X30	2650.000	1373.469	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL30X30	0.000	2667.418	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL30X30	1325.000	2608.453	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTEENTH	C67	COL30X30	2650.000	2614.554	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL30X30	0.000	4140.567	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL30X30	1325.000	4070.623	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTEENTH	C67	COL30X30	2650.000	4085.846	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL30X30	0.000	5627.075	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL30X30	1325.000	5549.136	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTEENTH	C67	COL30X30	2650.000	5569.570	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL30X30	0.000	7085.512	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL30X30	1325.000	7014.979	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRTEENTH	C67	COL30X30	2650.000	7036.211	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL40X40	0.000	2173.460	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL40X40	1325.000	2171.175	COMB10	0.000	ENVEQ	0.000	ENVEQ
TWELVETH	C67	COL40X40	2650.000	2168.889	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL40X40	0.000	2335.329	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL40X40	1325.000	2333.044	COMB10	0.000	ENVEQ	0.000	ENVEQ
ELEVENTH	C67	COL40X40	2650.000	2330.758	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL40X40	0.000	2501.578	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL40X40	1325.000	2498.006	COMB10	0.000	ENVEQ	0.000	ENVEQ
TENTH	C67	COL40X40	2650.000	2494.435	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL40X40	0.000	2668.550	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL40X40	1325.000	2664.979	COMB10	0.000	ENVEQ	0.000	ENVEQ
NINTH	C67	COL40X40	2650.000	2661.407	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C67	COL40X40	0.000	3260.940	COMB10	0.000	ENVEQ	0.000	ENVEQ
EIGTH	C67	COL40X40	1325.000	3012.210	COMB10	0.000	ENVEQ	0.000	ENVEQ

EIGTH	C67	COL40X40	2650.000	3081.188	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL40X40	0.000	4769.627	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL40X40	1325.000	4506.593	COMB10	0.000	ENVEQ	0.000	ENVEQ
SEVENTH	C67	COL40X40	2650.000	4595.807	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL40X40	0.000	6340.218	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL40X40	1325.000	6026.508	COMB10	0.000	ENVEQ	0.000	ENVEQ
SIXTH	C67	COL40X40	2650.000	6152.564	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL40X40	0.000	7874.747	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL40X40	1325.000	7497.172	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIFTH	C67	COL40X40	2650.000	7676.628	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL40X40	0.000	9431.758	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL40X40	1325.000	9065.442	COMB10	0.000	ENVEQ	0.000	ENVEQ
FOURTH	C67	COL40X40	2650.000	9211.309	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL40X40	0.000	10851.942	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL40X40	1325.000	10468.486	COMB10	0.000	ENVEQ	0.000	ENVEQ
THIRD	C67	COL40X40	2650.000	10763.525	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL40X40	0.000	12664.288	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL40X40	1325.000	12237.337	COMB10	0.000	ENVEQ	0.000	ENVEQ
SECOND	C67	COL40X40	2650.000	12420.963	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL50X50	0.000	4039.452	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL50X50	1325.000	4034.310	COMB10	0.000	ENVEQ	0.000	ENVEQ
FIRST	C67	COL50X50	2650.000	4029.167	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL50X50	0.000	4100.349	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL50X50	1250.000	4095.497	COMB10	0.000	ENVEQ	0.000	ENVEQ
GROUND	C67	COL50X50	2500.000	4090.645	COMB10	0.000	ENVEQ	0.000	ENVEQ

B-3 Foundation Reactions for HSC

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L O A D I N G C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
COMB1	ADD	DLTRANS	Static	1.0000

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S U P P O R T R E A C T I O N S

STORY	POINT	LOAD	FX	FY	FZ	MX	MY	MZ
BASE	99	COMB1	5.35	0.02	23350.34	-22.650	-19.475	-0.979
BASE	100	COMB1	-1.27	2.52	31855.02	-19.217	-20.690	-0.979
BASE	101	COMB1	456.23	439.42	4674.20	-37.869	43.057	0.046
BASE	104	COMB1	-461.85	446.87	4745.54	-38.505	-43.665	-0.047
BASE	105	COMB1	-1.90	-0.34	31949.34	0.693	-18.053	-0.842
BASE	106	COMB1	-5.64	-0.17	25671.65	5.346	-24.348	-1.132
BASE	108	COMB1	2.92	274.63	1593.75	-14.509	1.688	-0.129
BASE	145	COMB1	-0.24	2.17	15504.26	-2.159	-0.301	-0.004
BASE	144	COMB1	1.23	-422.38	2200.74	31.949	1.035	0.150
BASE	141	COMB1	-1.65	-430.15	2241.29	32.552	-1.330	-0.201
BASE	140	COMB1	0.30	1.71	15972.39	-1.556	0.064	-0.012
BASE	107	COMB1	-2.53	276.58	1600.93	-14.589	-1.449	0.111
BASE	109	COMB1	1.91	-4.79	2940.23	0.225	1.306	0.002
BASE	134	COMB1	-0.14	-2.59	17459.69	2.269	-0.231	-0.008
BASE	135	COMB1	1.28	395.71	2053.14	-29.886	1.072	-0.156
BASE	138	COMB1	-1.53	398.55	2067.61	-30.093	-1.241	0.18
BASE	139	COMB1	0.21	-1.59	18033.65	1.462	0.033	-0.012
BASE	98	COMB1	-1.74	-2.40	2981.49	0.152	-1.192	-0.001
BASE	133	COMB1	-0.83	1.95	33149.90	-21.928	-10.254	-1.132
BASE	132	COMB1	413.59	-0.98	6091.48	0.051	41.989	0.000

BASE	131	COMB1	-416.07	-0.45	6143.17	0.032	-42.280	0.000
BASE	130	COMB1	-0.66	0.65	33027.82	0.742	-10.233	-1.132
BASE	97	COMB1	-1.03	0.35	2963.68	-0.010	-0.595	0.000
BASE	111	COMB1	1.87	2.01	2892.41	-0.147	1.279	-0.002
BASE	126	COMB1	-0.15	2.10	16679.42	-2.103	-0.178	-0.008
BASE	127	COMB1	1.36	-393.42	2038.98	29.685	1.122	0.165
BASE	128	COMB1	-1.11	-397.89	2063.66	30.034	-0.959	-0.136
BASE	129	COMB1	0.15	1.61	16153.35	-1.538	0.121	-0.004
BASE	96	COMB1	-2.12	2.99	2991.64	-0.169	-1.506	0.001
BASE	112	COMB1	2.68	-267.14	1547.16	14.083	1.519	0.117
BASE	120	COMB1	-0.24	-2.05	16622.48	1.766	-0.233	-0.008
BASE	121	COMB1	1.43	411.21	2145.99	-31.131	1.177	-0.174
BASE	124	COMB1	-1.65	410.05	2140.03	-31.037	-1.323	0.200
BASE	125	COMB1	0.31	-2.58	16084.96	2.360	0.266	-0.012
BASE	95	COMB1	-2.50	-278.14	1610.53	14.677	-1.415	-0.109
BASE	113	COMB1	6.31	1.88	22655.52	-22.993	4.060	-0.979
BASE	114	COMB1	0.77	-0.48	37368.38	-21.631	3.449	-1.132
BASE	115	COMB1	440.97	-426.49	4527.30	36.723	41.668	-0.045
BASE	118	COMB1	-440.15	-426.03	4521.08	36.703	-41.582	0.045
BASE	119	COMB1	-0.13	0.12	37906.98	0.808	3.339	-1.132
BASE	94	COMB1	-6.02	0.01	25599.90	5.323	2.620	-1.132
BASE	86	COMB1	5.45	-3.57	5052.85	3.484	5.364	-0.001
BASE	87	COMB1	0.05	1.34	9507.94	-1.347	0.061	-0.002
BASE	88	COMB1	-0.33	0.88	9557.99	-0.908	-0.284	-0.004
BASE	89	COMB1	0.49	1.80	9075.59	-1.769	0.489	-0.002
BASE	90	COMB1	0.01	1.85	9203.32	-1.806	0.023	-0.002
BASE	91	COMB1	0.17	1.55	9109.12	-1.506	0.181	-0.002
BASE	92	COMB1	-1.48	1.20	8831.46	-1.163	-1.417	-0.002
BASE	93	COMB1	-6.55	-4.82	5887.15	4.516	-6.045	-0.008
BASE	407	COMB1	5.45	3.58	5064.01	-3.541	5.322	-0.001
BASE	409	COMB1	0.04	-1.39	9504.17	1.298	-0.021	-0.002
BASE	410	COMB1	-0.32	-0.87	9563.77	0.763	-0.415	-0.004
BASE	411	COMB1	0.49	-1.78	9104.04	1.700	0.413	-0.002
BASE	412	COMB1	0.01	-1.82	9226.05	1.748	-0.055	-0.002
BASE	413	COMB1	0.17	-1.52	9145.25	1.468	0.099	-0.002
BASE	414	COMB1	-1.49	-1.22	8835.78	1.184	-1.505	-0.002
BASE	408	COMB1	-6.54	4.82	5877.82	-4.445	-6.273	-0.008
BASE	136	COMB1	0.43	1.19	8607.55	-1.181	0.355	-0.004
BASE	137	COMB1	-0.30	1.27	9013.03	-1.241	-0.343	-0.004
BASE	24	COMB1	0.39	-1.23	8834.12	1.114	0.348	-0.004
BASE	33	COMB1	-0.43	-1.22	8804.98	1.126	-0.432	-0.004
BASE	566	COMB1	-250.54	4.23	4830.92	-3.929	-8.426	0.177
BASE	567	COMB1	-33.08	0.98	4882.44	-1.788	-0.988	-0.126
BASE	568	COMB1	5.20	3.58	4861.93	-3.305	0.897	0.010
BASE	569	COMB1	6.91	3.38	4836.38	-3.392	0.520	0.025
BASE	570	COMB1	1.61	2.86	4821.52	-3.051	0.067	0.000
BASE	571	COMB1	-3.68	3.42	4821.10	-3.428	-0.385	-0.026

BASE	572	COMB1	-2.05	3.63	4831.32	-3.351	-0.762	-0.009
BASE	573	COMB1	35.70	1.01	4836.53	-1.816	1.107	0.125
BASE	574	COMB1	249.67	4.18	4770.86	-3.883	8.413	-0.175
BASE	575	COMB1	-206.49	0.00	4354.90	0.000	-6.461	0.000
BASE	576	COMB1	-17.54	-0.01	4374.21	0.004	0.180	0.000
BASE	577	COMB1	11.07	-0.02	4340.36	0.011	1.164	0.000
BASE	578	COMB1	8.70	-0.01	4311.74	0.008	0.662	0.000
BASE	579	COMB1	0.83	0.00	4299.25	-0.004	0.038	0.000
BASE	580	COMB1	-7.01	0.01	4304.63	-0.018	-0.582	0.000
BASE	581	COMB1	-9.30	0.02	4325.99	-0.022	-1.077	0.000
BASE	582	COMB1	19.22	0.01	4352.30	-0.013	-0.087	0.000
BASE	583	COMB1	206.70	0.00	4325.52	-0.003	6.507	0.000
BASE	584	COMB1	-239.27	-4.06	4607.40	3.769	-7.983	-0.169
BASE	585	COMB1	-32.86	-1.04	4663.54	1.807	-1.010	0.122
BASE	586	COMB1	3.36	-3.61	4651.90	3.323	0.799	-0.009
BASE	587	COMB1	4.90	-3.40	4635.70	3.400	0.425	-0.026
BASE	588	COMB1	-0.14	-2.86	4629.73	3.042	-0.004	0.000
BASE	589	COMB1	-5.15	-3.41	4637.54	3.405	-0.431	0.026
BASE	590	COMB1	-3.48	-3.62	4655.42	3.328	-0.796	0.009
BASE	591	COMB1	33.05	-1.04	4668.34	1.807	1.034	-0.122
BASE	592	COMB1	239.97	-4.06	4612.91	3.770	8.045	0.169
BASE	607	COMB1	1.47	-236.93	3790.72	11.755	1.531	0.001
BASE	608	COMB1	-0.24	-39.04	3830.52	1.304	0.667	-0.080
BASE	609	COMB1	3.91	1.68	3807.23	-0.727	3.118	0.136
BASE	610	COMB1	3.91	-3.51	3814.98	0.817	3.120	-0.136
BASE	611	COMB1	-0.23	37.69	3845.82	-1.242	0.663	0.080
BASE	612	COMB1	1.45	237.25	3812.83	-11.790	1.485	0.000
BASE	601	COMB1	4.16	-266.11	3900.16	15.041	3.383	-0.139
BASE	602	COMB1	-0.32	-24.97	4008.66	2.112	0.667	0.084
BASE	603	COMB1	1.50	233.42	3985.86	-11.703	1.579	-0.001
BASE	604	COMB1	1.43	-241.26	4091.74	12.120	1.444	0.000
BASE	605	COMB1	-0.34	24.10	4122.88	-2.090	0.648	-0.089
BASE	606	COMB1	4.31	273.10	4018.71	-15.472	3.490	0.143
BASE	613	COMB1	-4.15	-265.80	3893.70	15.036	-3.368	0.139
BASE	614	COMB1	0.34	-25.10	4000.46	2.153	-0.672	-0.082
BASE	615	COMB1	-1.57	232.69	3976.06	-11.643	-1.706	0.002
BASE	616	COMB1	-1.57	-245.56	4165.22	12.373	-1.716	-0.002
BASE	617	COMB1	0.38	24.83	4194.45	-2.143	-0.681	0.086
BASE	618	COMB1	-4.35	278.22	4085.61	-15.755	-3.541	-0.146
BASE	619	COMB1	-1.39	-238.98	3834.68	11.858	-1.390	0.001
BASE	620	COMB1	0.22	-38.46	3871.48	1.251	-0.664	0.083
BASE	621	COMB1	-3.94	2.71	3844.23	-0.802	-3.149	-0.136
BASE	622	COMB1	-3.93	-3.52	3846.65	0.856	-3.142	0.137
BASE	623	COMB1	0.26	37.94	3876.08	-1.218	-0.683	-0.079
BASE	624	COMB1	-1.53	239.06	3840.99	-11.868	-1.643	0.001
BASE	143	COMB1	0.47	-9.19	9932.33	8.689	0.375	-0.004
BASE	142	COMB1	-0.43	-9.13	9451.04	8.647	-0.489	-0.004

BASE	122	COMB1	0.48	9.75	9790.50	-9.321	0.454	-0.004
BASE	123	COMB1	-0.42	9.75	9825.95	-9.308	-0.408	-0.004
BASE	1081	COMB1	0.25	-7.26	2897.50	0.126	0.076	0.018
BASE	1082	COMB1	-2.40	-53.26	2885.31	1.078	-1.561	0.016
BASE	1083	COMB1	-1.19	-194.81	2842.51	5.862	-0.161	-0.044
BASE	1084	COMB1	-1.10	198.94	2921.68	-6.000	-0.025	0.047
BASE	1085	COMB1	-2.43	52.68	2955.28	-1.056	-1.559	-0.017
BASE	1086	COMB1	0.26	4.93	2956.85	-0.060	0.082	-0.019
BASE	1087	COMB1	-0.45	-6.91	3002.52	0.121	-0.273	-0.019
BASE	1088	COMB1	2.25	-55.18	2994.92	1.131	1.430	-0.014
BASE	1089	COMB1	1.13	-202.68	2955.23	6.118	0.163	0.042
BASE	1090	COMB1	1.06	201.78	2938.89	-6.088	0.099	-0.042
BASE	1091	COMB1	2.22	55.24	2980.36	-1.133	1.425	0.015
BASE	1092	COMB1	-0.25	7.35	2990.04	-0.134	-0.091	0.017
BASE	110	COMB1	1.04	-1.58	2891.34	0.045	0.595	0.000
BASE	1172	COMB1	0.28	-7.43	2922.49	0.293	0.203	0.020
BASE	1173	COMB1	-1.66	-5.25	2906.96	0.184	-1.049	0.000
BASE	1174	COMB1	-0.21	-2.59	2897.54	0.065	-0.248	-0.017
BASE	1175	COMB1	-0.21	-0.55	2885.46	0.025	-0.246	0.017
BASE	1176	COMB1	-1.66	2.15	2882.85	-0.095	-1.050	0.000
BASE	1177	COMB1	0.26	4.43	2886.45	-0.208	0.186	-0.019
BASE	1178	COMB1	-0.18	-5.10	2971.79	0.222	-0.119	-0.019
BASE	1179	COMB1	1.66	-3.02	2964.17	0.117	1.061	0.000
BASE	1180	COMB1	0.21	-0.51	2962.44	0.004	0.244	0.017
BASE	1181	COMB1	0.22	1.21	2964.98	-0.025	0.248	-0.017
BASE	1182	COMB1	1.66	3.73	2969.27	-0.138	1.024	0.000
BASE	1183	COMB1	-0.39	5.77	2979.47	-0.242	-0.321	0.020
Summation	0, 0, Base	COMB1	0.00	0.00	1002737.25	20404401.075	-19132295.33	-21433.548

APPENDIX C

Analysis of Solid Slab and Beam for NSC and HSC

C-1 Analysis of solid slab for NSC and HSC

Two way slab systems using steel S-400 and Concrete C-30/C-95, the effective depth d is calculated as follows,

$$d \geq (0.4 + 0.6\,(f_{yk}/400))\, L_e/B_a$$

Where:

d is the effective slab depth

f_{yk} is the characteristic strength of the reinforcement (MPa)

L_e for two-way slabs, the shorter span length

B_a the appropriate constant term for slabs shown in Table C-1, and for slabs carrying partition walls likely to crack, shall be taken as $Ba \leq 150/L_0$

L_0 is the distance in meter between points of zero moments; and for a cantilever, twice the length to the face of the support.

Slab member	*Simply Supported*	*End Span*	*Interior Span*	*Cantilever*
Span 2:1	25	30	35	12
Span 1:1	35	40	45	10

Table C-1 B_a values for slabs

For two way slab systems using steel S-400 and Concrete C-30/C-95 ,the effective depth d is calculated as follows:-

Slab types used are S_1, S_2, S_3, S_4 and S_5 all with span ratio of 1:1

For end span slabs S_1, S_2 and S_5

d ≥ (0.4 + 0.6 (400/400)) 6000mm/40

≥ 150 mm

For simply supported slabs S_3,

d ≥ (0.4 + 0.6 (400/400)) 6000mm/35

≥ 171.4 mm

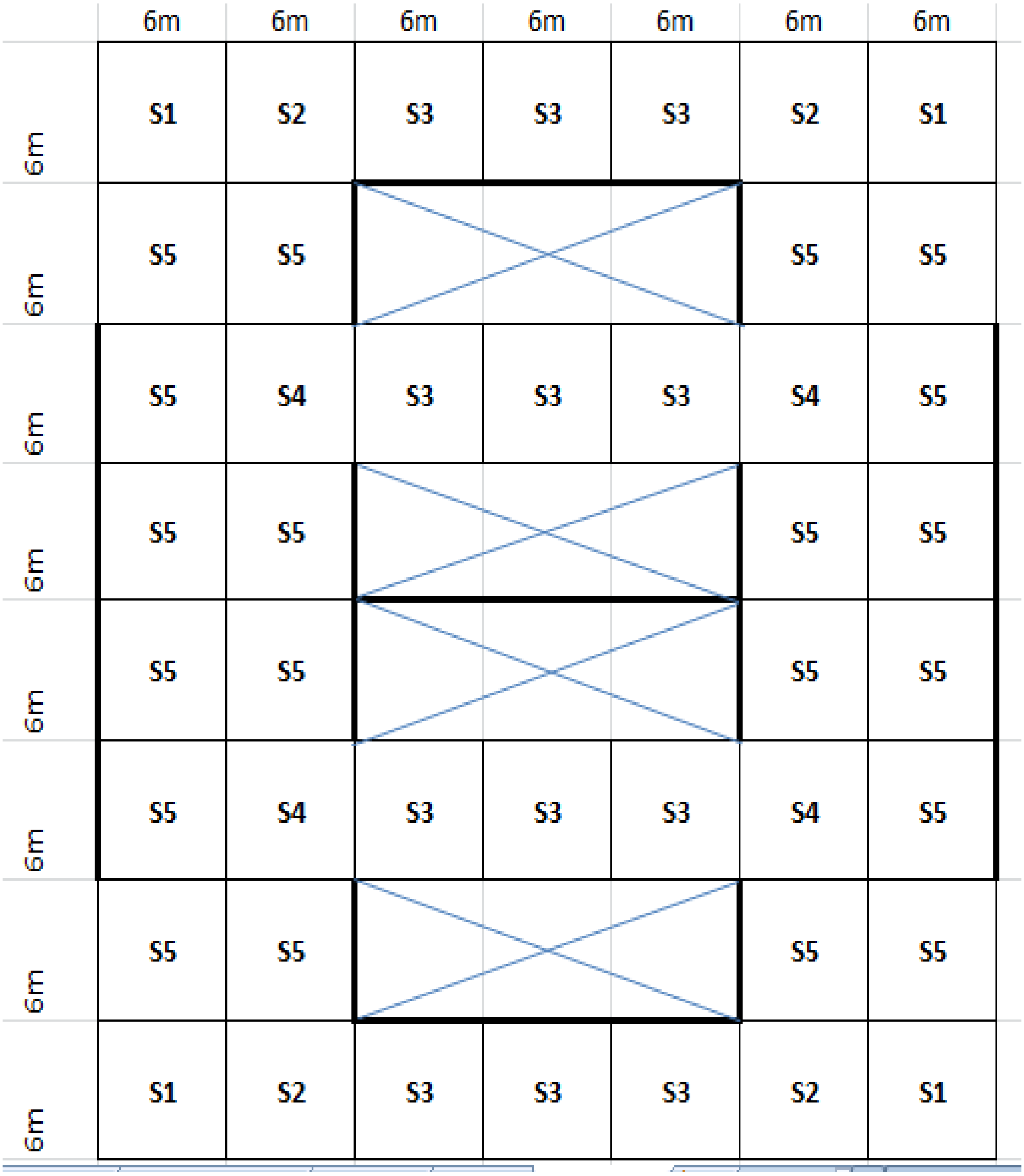


Figure C-1 Typical floor Slab Layout

For interior spans S_4,

$$d \geq (0.4 + 0.6\ (400/400))\ 6000mm/45$$
$$\geq 133.3\ mm$$

The final total slab depth D is calculated as:-

D = d + slab cover + (diameter of reinforcement to be used/2)

= 171.4 mm + 15 mm + (ø12mm/2)

=192.4 mm

Use D=200 mm for all typical floors

Slab Load Transfer using coefficients in EBCS-2

Ly/Lx = 6000mm/6000mm = 1 for all spans and the slab load transfer coefficients can be obtained from Table C-2.

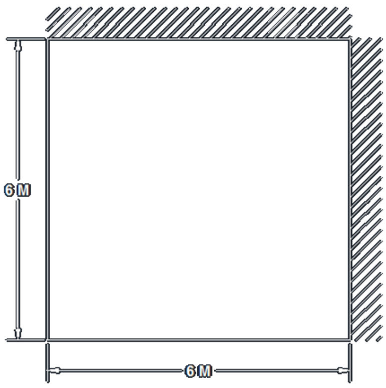


Figure C-2 Slab S_1

Type-4

a_{xs} = 0.047

a_{xf} = 0.036

a_{ys} = 0.047

a_{yf} = 0.036

β_a = 40 (end span)

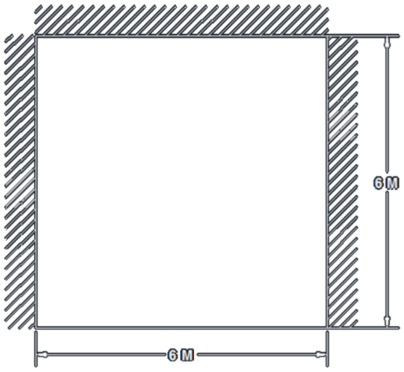


Figure C-3 Slab S_2

Type-3

a_{xs} = 0.039

a_{xf} = 0.030

a_{ys} = 0.039

a_{yf} = 0.030

B_a = 40 (end span)

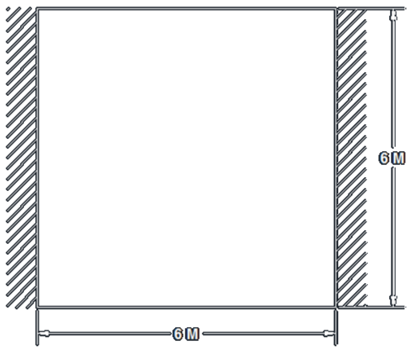


Figure C-4 Slab S_3

Type-6

a_{xs} = 0.034

a_{xf} = 0

a_{ys} = 0.045

a_{yf} = 0.034

B_a = 35 (simply supported)

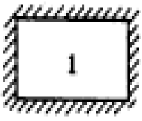
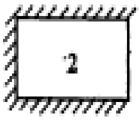
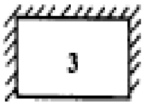
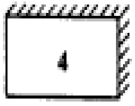
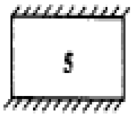
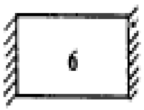
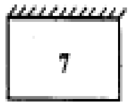
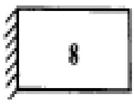
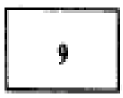
Support Condition	Coeff.	Values of L_y/L_x								Long span coefficients, α_{sy} and α_{sy}, for all values of L_y/L_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
 1	α_{sx} α_{sy}	0.032 0.024	0.037 0.028	0.042 0.032	0.046 0.035	0.050 0.037	0.053 0.040	0.059 0.044	0.063 0.048	0.032 0.024
 2	α_{sx} α_{sy}	0.039 0.029	0.044 0.033	0.048 0.036	0.052 0.039	0.055 0.041	0.058 0.043	0.063 0.047	0.067 0.050	0.039 0.029
 3	α_{sx} α_{sy}	0.039 0.030	0.049 0.036	0.056 0.042	0.062 0.047	0.068 0.051	0.073 0.055	0.082 0.062	0.089 0.067	0.039 0.030
 4	α_{sx} α_{sy}	0.047 0.036	0.056 0.042	0.063 0.047	0.069 0.051	0.074 0.055	0.078 0.059	0.087 0.065	0.093 0.070	0.047 0.036
 5	α_{sx} α_{sy}	0.046 0.034	0.050 0.038	0.054 0.040	0.057 0.043	0.060 0.045	0.062 0.047	0.067 0.050	0.070 0.053	- 0.034
 6	α_{sx} α_{sy}	- 0.034	- 0.046	- 0.056	- 0.065	- 0.072	- 0.078	- 0.091	- 0.100	0.045 0.034
 7	α_{sx} α_{sy}	0.057 0.043	0.065 0.048	0.071 0.053	0.076 0.057	0.081 0.060	0.084 0.063	0.092 0.069	0.098 0.074	- 0.044
 8	α_{sx} α_{sy}	- 0.044	- 0.054	- 0.063	- 0.071	- 0.078	- 0.084	- 0.096	- 0.105	0.058 0.044
 9	α_{sy}	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

Table C-2 Bending moment coefficients for rectangular panels supported on for sides

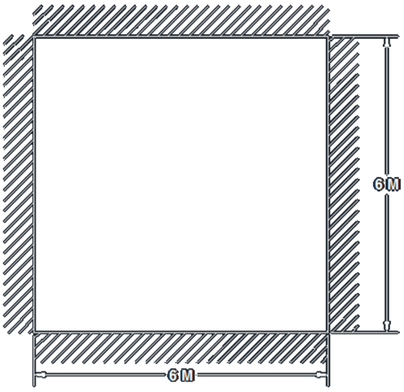


Figure C-5 Slab S_4

Type-1

a_{xs} = 0.032

a_{xf} = 0.024

a_{ys} = 0.032

a_{yf} = 0.024

B_a = 45 (interior span)

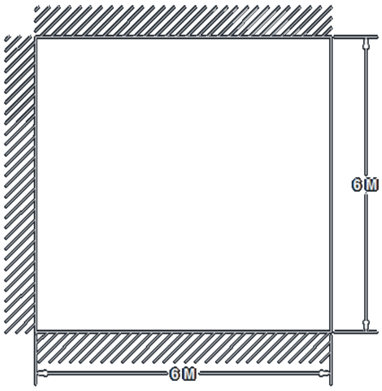


Figure C-6 Slab S_5

Type-2

a_{xs} = 0.047

a_{xf} = 0.036

a_{ys} = 0.047

a_{yf} = 0.036

B_a = 40 (end span)

Loadings:

Dead Load (DL)/Self Weight = (0.2 m) (25 kN/m^3) = 5 kN/m^2
Floor Finish /Ceramic clay tiling = (0.008 m) (21 kN/m^3) = 0.168 kN/m^2
Cement Screed = (0.042 m) (23 kN/m^3) = 0.09 kN/m^2
Partition load = 1 kN/m^2

- Total Dead Load = (5 kN/m^2 + 0.168 kN/m^2 + 0.09 kN/m^2 + 1kN/m^2)
  = 7.134 kN/m^2
- Live Load for apartment building (LL) = 2 kN/m^2

Total Design Load, P_d :-

$$P_d = 1.3DL + 1.6LL$$
$$= (1.3)\ (7.134\ kN/m2) + (1.6)\ (2\ kN/m2)$$
$$= 12.47 kN/m2$$

Slab Analysis (Moment Determination) :-

$$m_i = a_i\ P_d\ L_x^2$$

Where:

m_i is the design moment per unit width at the point of reference

a_i is the bending moment coefficients for rectangular panels supported on four panels

P_d is the total design load

L_x is the shorter span of a panel

1) S_1 (Type-4)

L_y/L_x=6000mm/6000mm=1

$$m_i = a_i\ P_d\ L_x^2$$
$$m_{xs} = (0.047)(12.47)(6^2)$$
$$= 21.1\ kNm/m$$

m_{ys} = (0.047)(12.47)(6^2)

= 21.1 kNm/m

m_{xf} = (0.036)(12.47)(6^2)

= 16.16 kNm/m

m_{yf} = (0.036)(12.47)(6^2)

= 16.16 kNm/m

2) S_2 (Type-3)

L_y/L_x = 6000mm/6000mm = 1

$m_i = a_i P_d L_x^2$

m_{xs} = (0.039)(12.47)(6^2)

= 17.5 kNm/m

m_{ys} = (0.039)(12.47)(6^2)

= 17.5 kNm/m

m_{xf} = (0.03)(12.47)(6^2)

= 13.47 kNm/m

m_{yf} = (0.03)(12.47)(6^2)

= 13.47 kNm/m

3) S_3 (Type-6)

L_y/L_x = 6000mm/6000mm = 1

$m_i = a_i P_d L_x^2$

m_{xs} = (0)(12.47)(6^2)

= 0 kNm/m

m_{ys} = (0.045)(12.47)(6^2)

= 20.2 kNm/m

m_{xf} = (0.034)(12.47)(6^2)

= 15.26 kNm/m

m_{yf} = (0.034)(12.47)(6^2)

= 15.26 kNm/m

4) S_4 (Type-1)

L_y/L_x = 6000mm/6000mm = 1

$m_i = a_i P_d L_x^2$

m_{xs} = (0.032)(12.47)(6^2)

= 14.36 kNm/m

m_{ys} = (0.032)(12.47)(6^2)

= 14.36 kNm/m

m_{xf} = (0.024)(12.47)(6^2)

= 10.77 kNm/m

m_{yf} = (0.024)(12.47)(6^2)

= 10.77 kNm/m

5) S_5 (Type-2)

L_y/L_x = 6000mm/6000mm = 1

$m_i = a_i P_d L_x^2$

m_{xs} = (0.039)(12.47)(6^2)

= 17.51 kNm/m

m_{ys} = (0.039)(12.47)(6^2)

= 17.51 kNm/m

m_{xf} = (0.029)(12.47)(6^2)

=13.02 kNm/m

m_{yf} = (0.029)(12.47)(6^2)

=13.02 kNm/m

Support Moments Adjustment:-

- Panel S_1 and S_2

  S_1 :- m_{xs} = 21.1 kNm/m

  m_{ys} = 21.1 kNm/m

  S_2 :- m_{xs} = 17.5 kNm/m

  m_{ys} = 17.5 kNm/m

  Δm = (21.1 kNm/m) - (17.5 kNm/m) = 3.6 kNm/m

  m_{large} = 21.1 kNm/m

  0.2 m_{large} = 4.22 kNm/m

$\Delta m < 0.2\ m_{large}$ OK!

m_{avg} = ((21.1 kNm/m) + (17.5 kNm/m))/2

= 19.3kNm/m

- Panel S_2 and S_3

S_2 :- m_{ys} = 17.5 kNm/m

S_3 :- m_{ys} = 20.2 kNm/m

Δm = (20.2 kNm/m) - (17.5 kNm/m) = 2.7 kNm/m

m_{large} = 20.2 kNm/m

0.2 m_{large} = 4.04 kNm/m

$\Delta m < 0.2\ m_{large}$ OK!

m_{avg} = ((20.2 kNm/m) + (17.5 kNm/m))/2

=18.85 kNm/m

- Panel S_1 and S_5

S_1 :- m_{xs} = 21.1 kNm/m

m_{ys} = 21.1 kNm/m

S_5 :- m_{xs} = 17.5 kNm/m

m_{ys} = 17.5 kNm/m

Δm = (21.1 kNm/m) - (17.5 kNm/m) = 3.6 kNm/m

m_{large} = 21.1 kNm/m

0.2 m_{large} = 4.22 kNm/m

$\Delta m < 0.2\ m_{large}$ OK!

m_{avg} = ((21.1 kNm/m) + (17.5 kNm/m))/2

=19.3 kNm/m

- Panel S_4 and S_5

S_4 :- m_{xs} =14.36 kNm/m

m_{ys} = 14.36 kNm/m

S_5 :- m_{xs} = 17.5 kNm/m

m_{ys} = 17.5 kNm/m

Δm = (17.5 kNm/m) - (14.36 kNm/m) = 3.15 kNm/m

m_{large} = 17.51 kNm/m

0.2 m_{large} = 3.5 kNm/m

$\Delta m < 0.2\ m_{large}$ OK!

m_{avg} = ((17.51 kNm/m) + (14.36 kNm/m))/2

=15.9 kNm/m

- Panel S_3 and S_4

  S_3 :- m_{ys} = 20.2 kNm/m

  S_4 :- m_{ys} = 14.36 kNm/m

  Δm = (20.2 kNm/m) - (14.36 kNm/m) = 5.84 kNm/m

  m_{large} = 20.2 kNm/m

  0.2 m_{large} = 4.04 kNm/m

  $\Delta m > 0.2\ m_{large}$ CHECK!

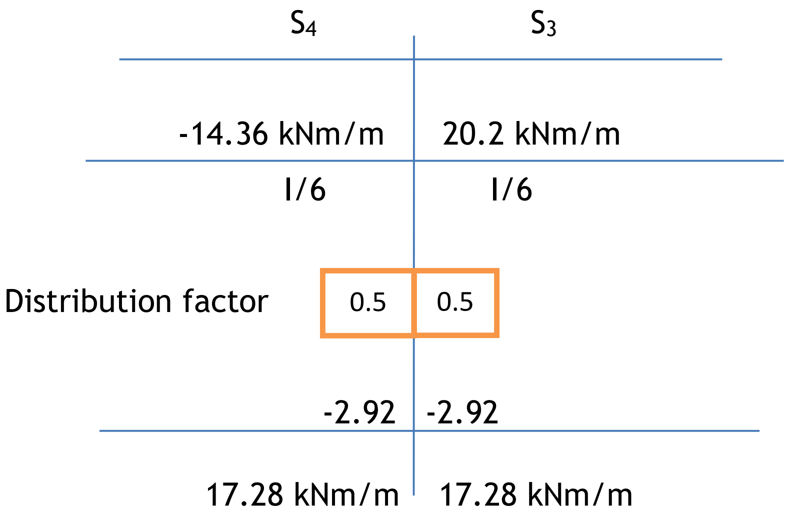


Figure C-7 Moment distribution for slabs S_3 and S_4

14.36 kNm/m →17.28 kNm/m (no adjustment is needed)

20.2 kNm/m →17.28 kNm/m (adjustment is needed)

To overcome decrease of support moment from 20.2 kNm/m to 17.28 kNm/m, adjust field moments by increasing m_{xf} and m_{yf} to allow for change of support moments.

Panel S_4 = 14.36 kNm/m (no adjustment)

Panel S_3 = 17.28 kNm/m (adjustment needed)

Δms = (20.2 kNm/m) –(17.28 kNm/m) = 2.92 kNm/m

According to Table C-3,

For S_3 type of slab

Ly/Lx = 1

Factors for adjusting span moments can be taken as ,C_x = 0.38 and C_y = 0.28

$\Delta m_{xf} = C_x \Delta ms$ = (0.38) (2.92 kNm/m) = 1.1096 kNm/m

$\Delta m_{yf} = C_y \Delta ms$ = (0.28) (2.92 kNm/m) = 0.8176 kNm/m

Adjust span moments on S_3 since support moment is decreased from 20.2 kNm/m →17.28 kNm/m

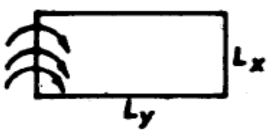
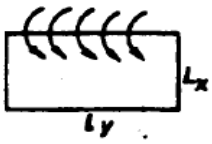
L_y/L_x				
	C_x	C_y	C_x	C_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481	0.251
2.0	0.238	0.055	0.484	0.248

Table C-3 Factors for adjusting span moments m_{xf} and m_{yf}

Initial: - m_{xf} = 15.26 kNm/m

m_{yf} = 15.26 kNm/m

Final: - m_{xf} = (15.26 kNm/m) + (1.1096 kNm/m) = 16.37 kNm/m

m_{yf} = (15.26 kNm/m) + (0.8176 kNm/m) = 16.08

$m_{x3} = m_{x4}$ = 17.28 kNm/m

Shear Force Transfer on Beams:-

$$V_s=\beta_v P_d L_x$$

Where:

$V_x = \beta_{vx} P_d L_x$

$V_y = \beta_{vy} P_d L_y$

β_v Load transfer (Shear force coefficient) can be obtained from Table C-4

Panel S_1 (type -4)

Ly/Lx = 6000mm/6000mm = 1

	β_{vx}	β_{vy}
Continuous	0.4	0.4
Discontinuous	0.26	0.26

$V_x = \beta_{vx} P_d L_x$

= (0.4)(12.47 kN/m^2)(6m) cont.

= 29.93 kNm/m

$V_x = \beta_{vx} P_d L_x$

= (0.26) (12.47 kN/m^2) (6m) discont.

= 19.45 kNm/m

$V_y = \beta_{vy} P_d L_y$

= (0.4) (12.47 kN/m^2) (6m) cont.

= 29.93kNm/m

$V_y = \beta_{vy} P_d L_y$

= (0.26) (12.47 kN/m^2) (6m) discont.

= 19.45kNm/m

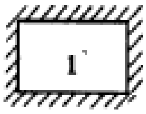
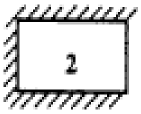
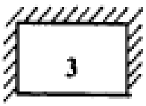
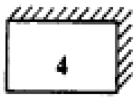
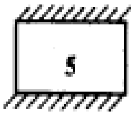
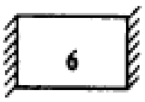
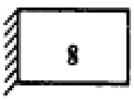
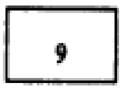
Type of panel and location	Edge	β_{vx} for values of L_y/L_x								β_{vy}
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
	Continuous	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinous	-	-	-	-	-	-	-	-	0.24
	Continuous	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
	Discontinous	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
	Continuous	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
	Discontinous	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
	Continuous	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
	Discontinous	-	-	-	-	-	-	-	-	0.26
	Continuous	-	-	-	-	-	-	-	-	0.40
	Discontinous	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
	Continuous	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
	Discontinous	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.30
	Continuous	-	-	-	-	-	-	-	-	0.45
	Discontinous	0.30	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
	Discontinous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

Table C-4 Shear force coefficients for uniformly loaded rectangular panels supported on four sides

Plane S_3 (type-6)

Ly/Lx = 6000mm/6000mm = 1

	β_{vx}	β_{vy}
Continuous	-	0.4
Discontinuous	0.26	-

$V_x = \beta_{vx} P_d L_x$

$= (0)\ (12.47\ kN/m^2)\ (6m)$ cont.

$= 0$ kNm/m

$V_x = \beta_{vx} P_d L_x$

$= (0.26)\ (12.47\ kN/m^2)\ (6m)$ discont.

$= 19.45$ kNm/m

$V_y = \beta_{vy} P_d L_y$

$= (0.4)\ (12.47\ kN/m^2)\ (6m)$ cont.

$= 29.93$ kNm/m

$V_y=\beta_{vy} P_d L_y$

$=(0)(12.47kN/m^2)(6m)$ discont.

=0kNm/m

Panel S_4 (type -1)

Ly/Lx = 6000mm/6000mm = 1

	β_{vx}	β_{vy}
Continuous	0.33	0.33
Discontinuous	-	-

$V_x = \beta_{vx}\, P_d\, L_x$

= (0.33) (12.47 kN/m^2) (6m)   cont.

= 24.69 kNm/m

$V_x = \beta_{vx}\, P_d\, L_x$

= (0) (12.47 kN/m^2) (6m) discont.

=0 kNm/m

$V_y = \beta_{vy}\, P_d\, L_y$

= (0.33) (12.47 kN/m^2) (6m)   cont.

= 24.69 kNm/m

$V_y = \beta_{vy}\, P_d\, L_y$

= (0) (12.47 kN/m^2) (6m) discont.

= 0 kNm/m

Panel S_5 (type -2)

Ly/Lx = 6000mm/6000mm = 1

	β_{vx}	β_{vy}
Continuous	0.36	0.36
Discontinuous	-	0.24

$V_x = \beta_{vx}\, P_d\, L_x$

= (0.36) (12.47 kN/m^2) (6m)   cont.

= 26.94 kNm/m

$V_x = \beta_{vx}\, P_d\, L_x$

= (0) (12.47 kN/m^2) (6m) discont.

= 0 kNm/m

$V_y = β_{vy} P_d L_y$

= (0.36) (12.47 kN/m^2) (6m) cont.

= 26.94 kNm/m

$V_y = β_{vy} P_d L_y$

= (0.24) (12.47 kN/m^2) (6m) discont.

=17.96 kNm/m

Panel S_2 (type -3)

Ly/Lx = 6000mm/6000mm = 1

	$β_{vx}$	$β_{vy}$
Continuous	0.36	0.36
Discontinuous	0.24	-

$V_x = β_{vx} P_d L_x$

= (0.36) (12.47 kN/m^2) (6m) cont.

= 26.94 kNm/m

$V_x = β_{vx} P_d L_x$

= (0.36) (12.47 kN/m^2) (6m) discont.

= 26.94 kNm/m

$V_y = β_{vy} P_d L_y$

=(0.24) (12.47 kN/m^2) (6m) cont.

= 17.96 kNm/m

$V_y = \beta_{vy} P_d L_y$

= (0) (12.47 kN/m^2) (6m) discont.

= 0kNm/m

Using HCB for walling with unit weight of 17 kN/m^3

- External walls with thickness 0.2m and floor height of 3m,
  = (0.2m) (3m) (17 kN/ m^3)
  = 10.2 kN/m
- Internal walls with thickness 0.15m and height of 3m
  =(0.15m) (3m) (17 kN/ m^3)
  = 7.65 kN/m

C-2 Analysis of beam for NSC and HSC

The effective depth d is calculated as follows,

$$d \geq (0.4 + 0.6\ (f_{yk}/400))\ L_e/B_a$$

Where:

d is the effective beam depth

f_{yk} is the characteristic strength of the reinforcement (MPa)

L_e the effective span

B_a the appropriate constant term for beams shown in Table C-5.

Beam member	Simply Supported	End Spans	Interior supports	Cantilevers
B_a values	20	24	28	10

Table C-5 B_a values for beams

For simply supported beams,

$d \geq (0.4 + 0.6\ (400/400))\ 6000mm/20$

≥ 300 mm

For end span beams,

$d \geq (0.4 + 0.6\ (400/400))\ 6000mm/24$

≥ 250 mm

For interior beams,

$d \geq (0.4 + 0.6\ (400/400))\ 6000mm/28$

≥ 214.3 mm

The final floor beam depth D is calculated as

D = d + beam cover + diameter of stirrup to be used + (diameter of reinforcement to be used/2)

= 300mm + 25mm + ø8mm + (ø14mm/2)

=340 mm

Use D = 350mm

APPENDIX D

Analysis of Wind Pressure acting on NSC and HSC buildings

Building type:-Apartment with internal partitions

Building location:- Addis Ababa on a site surrounded by buildings of similar height

Terrain category:- IV

Reference wind velocity V_{ref} = 22m/s

Air density ρ = 0.94 kg/m^3 for Addis Ababa

Total Height of the building H = 75m

- Reference wind pressure (q_{ref})

  $q_{ref} = (\rho)(V_{ref})/2$

  $= ((0.94 \text{ kg/m}^3)(22\text{m/s}))/2$

  $= 0.23 \text{ kN/m}^2$

- Reference height (Z_e)

  H = 75m

  Building breadth b = 42m

A) Wind direction:- Y

According to Figure D-1 below, wind force in the Y-direction,

If $H < b$, $Z_e = h$

$b < H < 2b$, $Z_e = b$ and $Z_e = H$

$H > 2b$, Z_e = b from the ground and $Z_e = b$ from the top

2b = 84m

Since $b < H < 2b$

42m < 75m < 84m

$Z_e = H_1$ = 42m and $Z_e = H_2$ = 75m

The building is divided into two parts,

Lower part extends from the ground level to the height $H_1 = b = Z_e$ = 42m and upper part extends from the ground level to the height $H_2 = Z_e$ = 75m

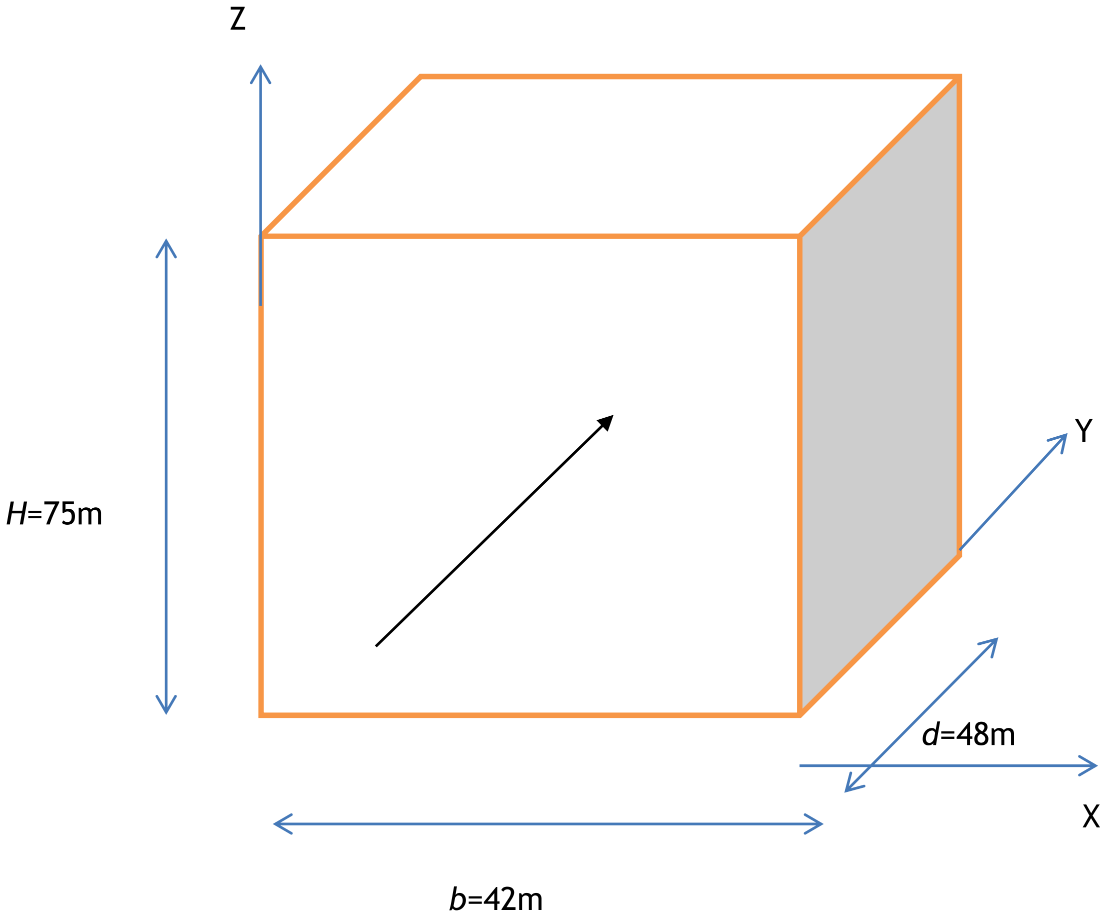


Figure D-1 Wind force in the Y-direction

- Exposure coefficient $C_e(Z)$

  $C_e(Z)$ is $f\ (C_t\ C_r, K_T)$

  a) Topographic coefficient, $C_t(Z)$

     Assuming a flat terrain $\Phi = H/L_u$

     Φ = Upwind slope

     L_u = Actual length of an upwind slope

     $C_t(Z) = C_t = 1$ for $\Phi < 0.05$

  b) Roughness coefficient $C_r(Z)$

     $C_r(Z) = K_T\ ln(Z/Z_o)\ for\ Z_{min} \leq Z \leq 200m$ and

     $C_r(Z) = C_r(Z_{min})\ for\ Z_e < Z_{min}$

     K_T = terrain factor

     Z_o = roughness length

     Z_{min} = the minimum height

For terrain category IV,

$K_T = 0.24$

$Z_o = 1m$

$Z_{min} = 16m$

Thus,

For the lower part of the building

$Z_e = 42m = Z$

$Z_e = 42m > Z_{min} = 16m$ i.e, $Z_{min} \leq Z \leq 200m$, $16m \leq 42m \leq 200m$

$C_r(Z) = K_T \ln(Z/Z_o)$

$= (0.24) (\ln (42m/1m))$

$= 0.897$

For the upper part of the building

$Z_e = 75m = Z$

$Z_e = 75m > Z_{min} = 16m$ i.e, $Z_{min} \leq Z \leq 200m$, $16m \leq 75m \leq 200m$

$C_r(Z)= K_T \ln(Z/Z_o)$

$= (0.24) (\ln (75m/1m))$

$= 1.04$

Thus,

For the lower part,

Exposure coefficient, C_e

$C_e(Z) = C_r^2(Z) C_t^2(Z) (1 + (7K_T/C_r(Z) C_t(Z)))$

$C_e(42m) = ((0.897)^2) (1^2) ((1 + (7) (0.24)/(0.897) (1))$

$= 2.3$

For the upper part,

$C_e(75m) = (1.04)^2) (1^2) ((1 + (7) (0.24)/(0.04) (1))$

$= 2.83$

External Pressure coefficient (C_{pe}):-

From Figures D-2 and D-3, it could be observed that

e = b or 2H whichever is smaller

b = 42m and 2H = (2) (75m) = 150m

Thus, e = 42m and in our case, d > e i.e 48m > 42m e/5 = 42/5 = 8.4m

Plan:-

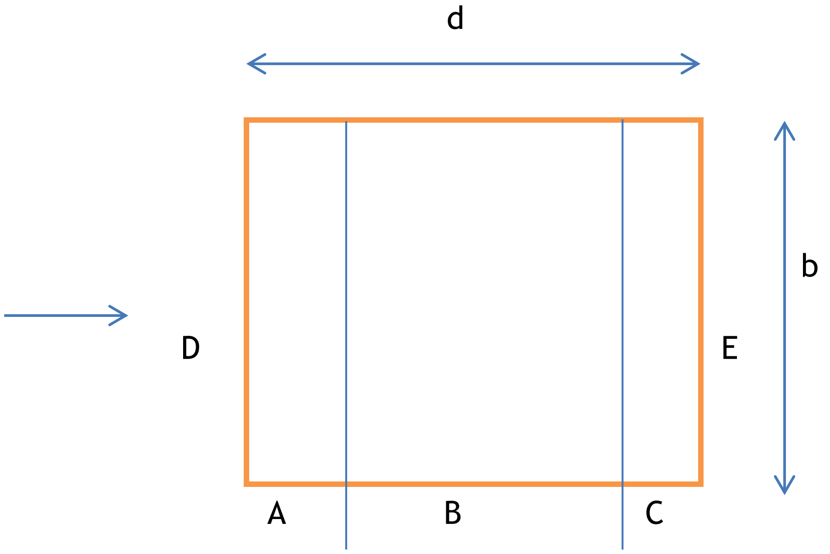


Figure D-2 Plan view of building for wind in Y-Direction

Elevation:-

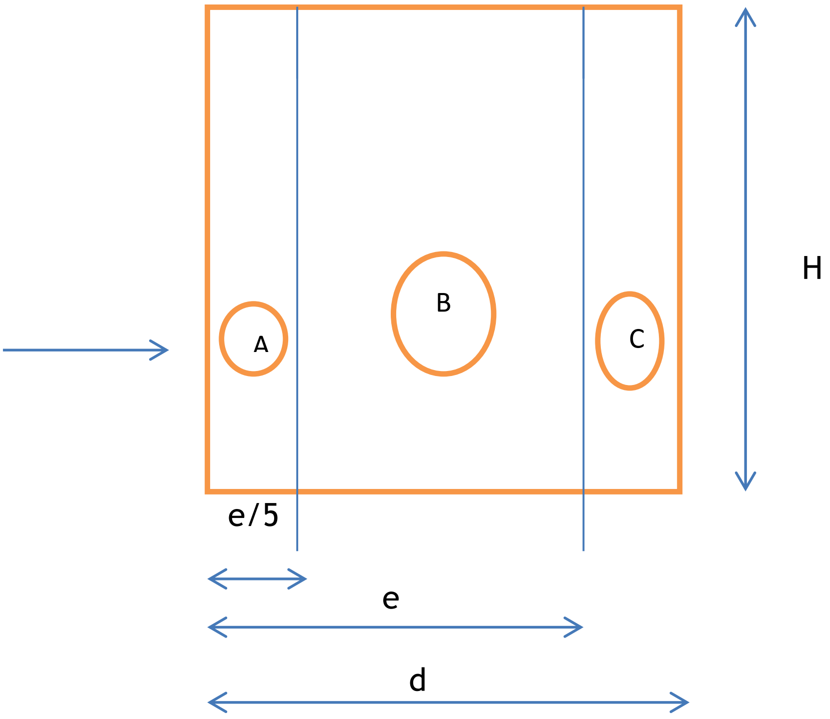


Figure D-3 Elevation View of building for wind in Y-Direction

Areas :- A = (8.4m) (75m) = 630m^2

B = (33.6m) (75m) = 2,520m^2

C = (6m) (75m) = 450m^2

$C_{pe} = C_{pe,10}$ for A ≥ 10m^2 ,

$C_{pe} = C_{pe,1}$ for A ≤ 1m^2

$C_{pe} = C_{pe,1} + (C_{pe,10} - C_{pe,1}) (log^{A}{}_{10})$ for 1m^2 < A < 10m^2

Zone	A		B,B*		C		D		E	
d/h	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$
≤1	- 1.0	- 1.3	- 0.8	- 0.1	- 0.5		+0.8	+1.0	- 0.3	
≥4	- 1.0	- 1.3	- 0.8	- 0.1	- 0.5		+0.6	+1.0	- 0.3	

Table D-1 External pressure coefficients for vertical walls of rectangular plan buildings

d/H = 48m/75m = 0.64

Zone	$C_{pe,10}$
A	-1
B	-0.8
C	-0.5
D	0.8
E	-0.3

- External wind pressure

  $W_e = q_{ref}\ C_e(Z_e)\ C_{pe}$, where q_{ref} = 0.23 and $C_e(Z_e)$ = 2.3 and 2.83

Zone	$C_{pe} = C_{pe,10}$	$We=(0.23)(2.3)(C_{pe})$	$We=(0.23)(2.83)(C_{pe})$
A	-1	-0.53	-0.651
B	-0.8	-0.423	-0.521
C	-0.5	-0.265	-0.326
D	0.8	0.423	0.521
E	-0.3	-0.159	-0.196

Roof

Flat roof with sharp eaves

H = 75m

e/4 = 42m/4m = 10.5

e/2 = 42m/2m = 21

e/10 = 42m/10m = 4.2

Plan:-

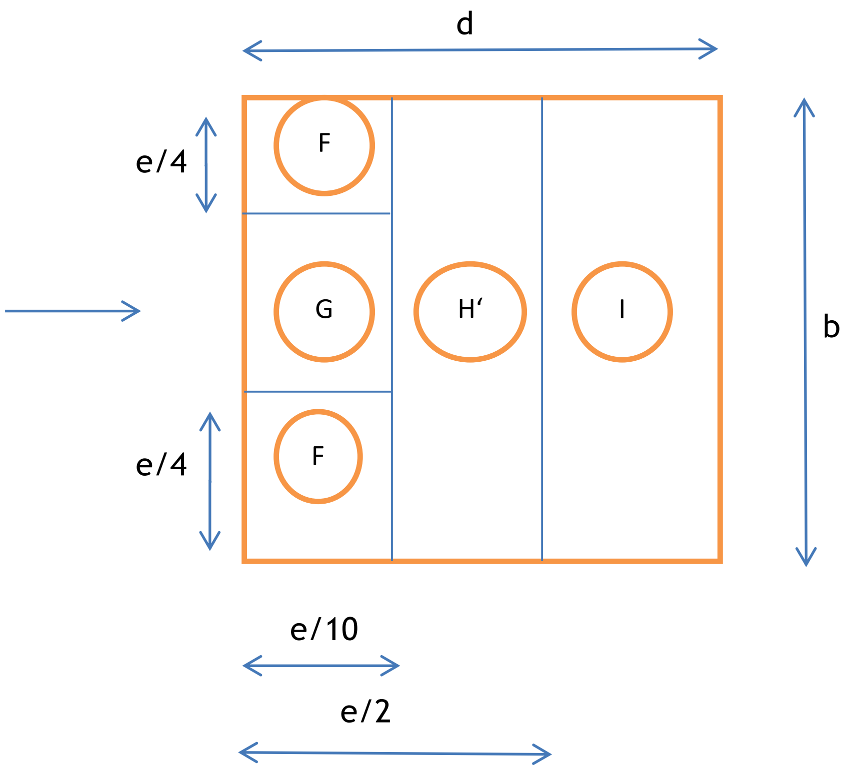


Figure D-4 Plan view of roof for wind in Y-Direction

Areas: F = (4.2m) (10.5m) = 44.1m^2

G = (4.2m) (21m) = 88.2m^2

H' = (16.8m) (42m) = 701.4m^2

I = (27m) (42m) = 1,134m^2

Since, $C_{pe} = C_{pe,10}$ for A ≥10m^2

At H = 75m, $W_e = q_{ref}\ C_e(Z_e)\ C_{pe}$, where q_{ref} = 0.23 and $C_{e}(Z_e)$ = 2.83

		Zone							
		F		G		H			I
		$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$
Sharp eaves		- 1.8	- 2.5	- 1.2	- 2.0	- 0.7	- 1.2	± 0.2	
with parapets	H_p/h = 0.025	- 1.6	- 2.2	- 1.1	- 1.8	- 0.7	- 1.2	± 0.2	
	H_p/h = 0.05	- 1.4	- 2.0	- 0.9	- 1.6	- 0.7	- 1.2	± 0.2	
	H_p/h = 0.10	- 1.2	- 1.8	- 0.8	- 1.4	- 0.7	- 1.2	± 0.2	
Curved eaves	r/h = 0.05	- 1.0	- 1.5	- 1.2	- 1.8	- 0.4		± 0.2	
	r/h = 0.10	- 0.7	- 1.2	- 0.8	- 1.4	- 0.3		± 0.2	
	r/h = 0.20	- 0.5	- 0.8	- 0.5	- 0.8	- 0.3		± 0.2	
mansard eaves	$\alpha = 30°$	- 1.0	- 1.5	- 1.0	- 1.5	- 0.3		± 0.2	
	$\alpha = 45°$	- 1.2	- 1.8	- 1.3	- 1.9	- 0.4		± 0.2	
	$\alpha = 60°$	- 1.3	- 1.9	- 1.3	- 1.9	- 0.5		± 0.2	

Table D-2 External pressure coefficients for flat roofs

Zone	$C_{pe,10}$	$We=(0.23)(2.83)(C_{pe})$
F	-1.8	-1.172
G	-1.2	-0.781
H'	-0.7	-0.456
I	+0.2 and -0.2	0.1302 and -0.1302

Elevation:-

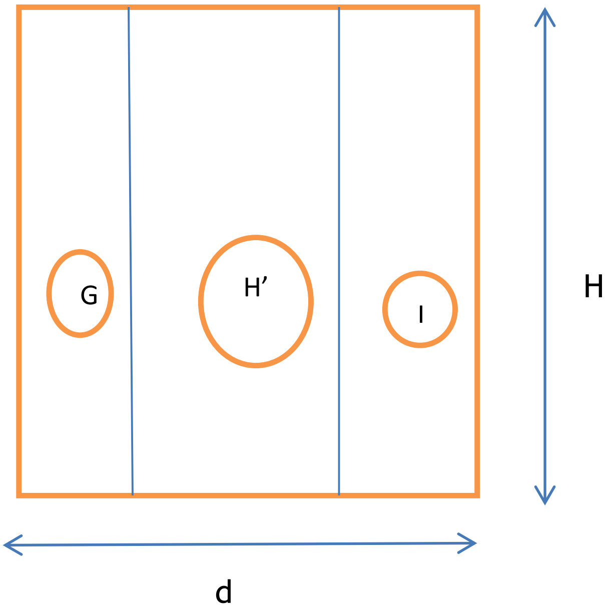


Figure D-5 Elevation view for wind in Y-Direction for roof analysis

B) Wind direction:- X

- Reference wind pressure (q_{ref})

  $q_{ref} = (\rho)(V_{ref})/2$

  $= ((0.94\text{kg/m}^3)\ (22\text{m/s}))/2$

  $= 0.23\text{kN/m}^2$

- Reference height (Z_e)

  H =75m

  Building breath b = 48m

  If H < b, Z_e = H

  b < H < 2b, Z_e = b and Z_e = H

  H > 2b, Z_e = b from the ground and Z_e = b from the top

  2b = 96m

  Since b < H < 2b

  48m < 75m < 84m

  $Z_e = H_1$ = 48m and $Z_e = H_2$ = 75m

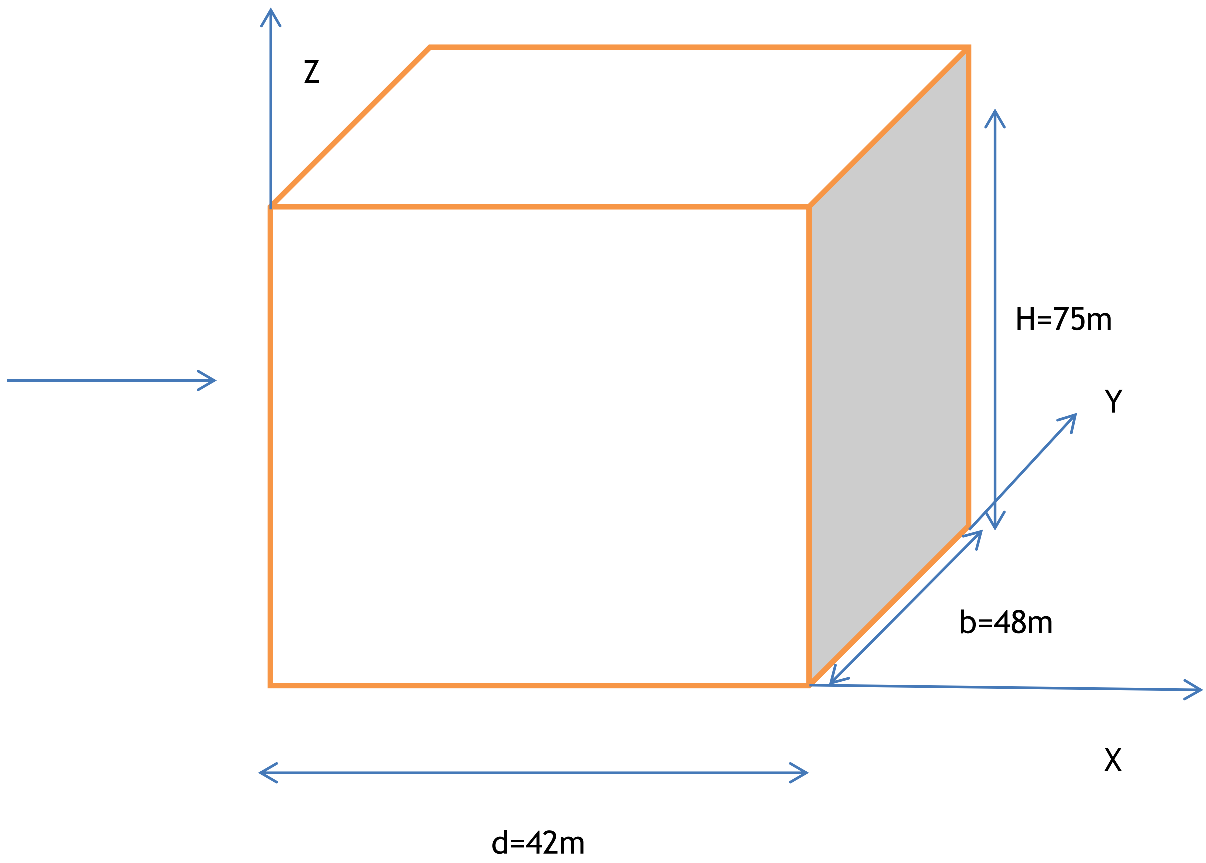


Figure D-6 Wind direction in the X-direction

The building is divided into two parts,

Lower part extends from the ground level to the height H_1 = b = Z_e = 48m and upper part extends from the ground level to the height H_2 = Z_e = 75m

- Exposure coefficient $C_e(Z)$

  $C_e(Z)$ is $f\ (C_t\ C_r, K_T)$

  a) Topographic coefficient, $C_t(Z)$

  Assuming a flat terrain $\Phi = H/L_u$

  Φ = Upwind slope

  L_u = Actual length of an upwind slope

  $C_t(Z) = C_t = 1$ for $\varnothing < 0.05$

  b) Roughness coefficient $C_r(Z)$

  $C_r(Z) = K_T\ ln\ (Z/Z_o)$ for $Z_{min} \leq Z \leq 200$m and

  $C_r(Z) = C_r\ (Z_{min})$ for $Z_e < Z_{min}$

c) K_T = terrain factor

Z_o = roughness length

Z_{min} = the minimum height

For terrain category IV,

$K_T = 0.24$

$Z_o = 1m$

$Z_{min} = 16m$

Thus,

For the lower part:

$Z_e = 48m = Z$

$Z_e = 48m > Z_{min} = 16m$ i.e, $Z_{min} \leq Z \leq 200m$, $16m \leq 48m \leq 200m$

$C_r(Z) = K_T \ln(Z/Z_o)$

$= (0.24) (\ln (48m/1m))$

$= 0.929$

For the upper part:

$Z_e = 75m = Z$

$Z_e = 75m > Z_{min} = 16m$ i.e, $Z_{min} \leq Z \leq 200m$, $16m \leq 75m \leq 200m$

$C_r(Z) = K_T \ln(Z/Z_o)$

$= (0.24) (\ln (75m/1m))$

$= 1.04$

- Exposure coefficient, C_e

  $C_e(Z) = C_r^2(Z)C_t^2(Z) (1 + (7K_T/C_r(Z)C_t(Z)))$

  Thus,

  For the lower part,

  $C_e(48m) = (0.929)^2 (1^2) (1+(7) (0.24)/(0.929)(1))$

  $= 2.424$

  For the upper part,

  $C_e(75m) = (1.04)2(1^2) (1+(7) (0.24)/(0.04)(1))$

  $= 2.83$

- External Pressure coefficient (C_{pe})

e = b or 2H whichever is smaller

$b = 48m$ and $2H = (2)(75m) = 150m$

Thus, $e = 48m$ and in our case, $d < e$ i.e $42m < 48m$

$e/5 = 48/5 = 9.6m$

Plan :-

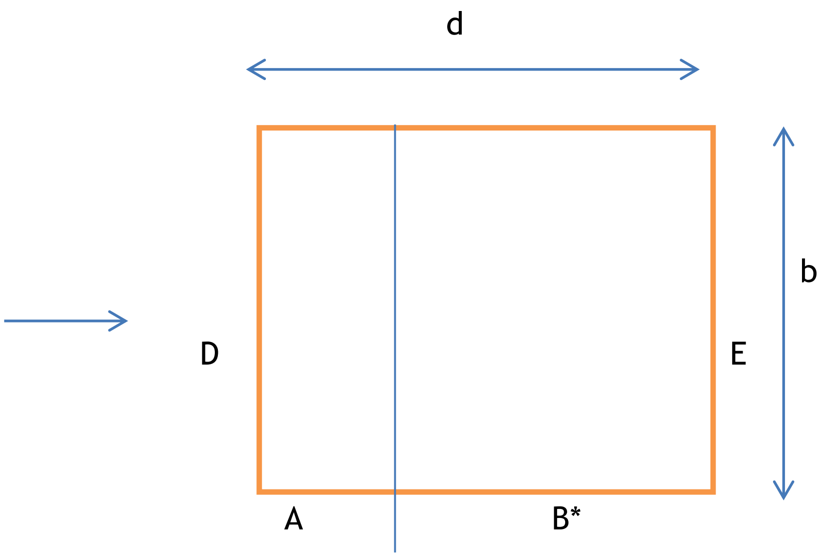


Figure D-7 Plan view of building for wind in X-Direction

Areas:- $A = (9.6m)(75m) = 720m^2$

$B^* = (32.4m)(75m) = 2{,}430m^2$

$C_{pe} = C_{pe,10}$ for $A \geq 10m^2$,

$C_{pe} = C_{pe,1}$ for $A \leq 1m^2$

$C_{pe} = C_{pe,1} + (C_{pe,10} - C_{pe,1})(log^{A}{}_{10})$ for $1m^2 < A < 10m^2$

From Table D -1

$d/H = 42m/75m = 0.56$

Elevation:-

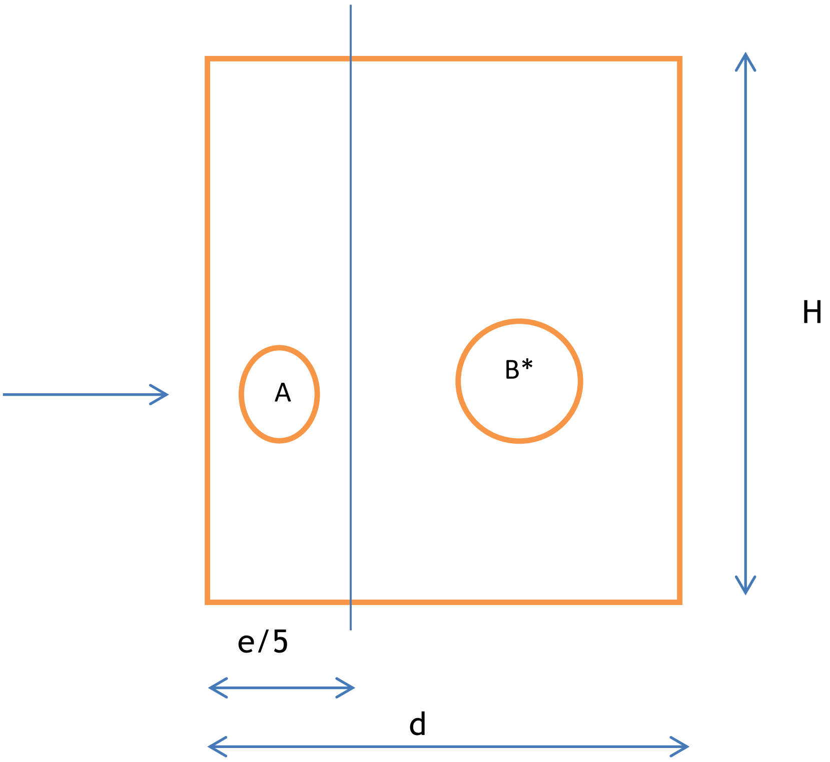


Figure D-8 Elevation view of building for wind in X-Direction

Zone	$C_{pe,10}$
A	-1
B*	-0.8
C	-0.5
D	0.8
E	-0.3

- External wind pressure

$W_e = q_{ref}\ C_e(Z_e)C_{pe}$,   where q_{ref} = 0.23 and $C_e(Z_e)$ = 2.42 and 2.83

Zone	$C_{pe} = C_{pe,10}$	$We=(0.23)(2.42)(C_{pe})$	$We=(0.23)(2.83)(C_{pe})$
A	-1	-0.557	-0.651
B	-0.8	-0.445	-0.521
C	-0.5	-0.278	-0.326
D	0.8	0.445	0.521
E	-0.3	-0.167	-0.196

Roof

Flat roof with sharp eaves

H = 75m

e/4 = 48m/4m = 12

e/2 = 48m/2m = 24

e/10 = 48m/10m = 4.8

Plan:-

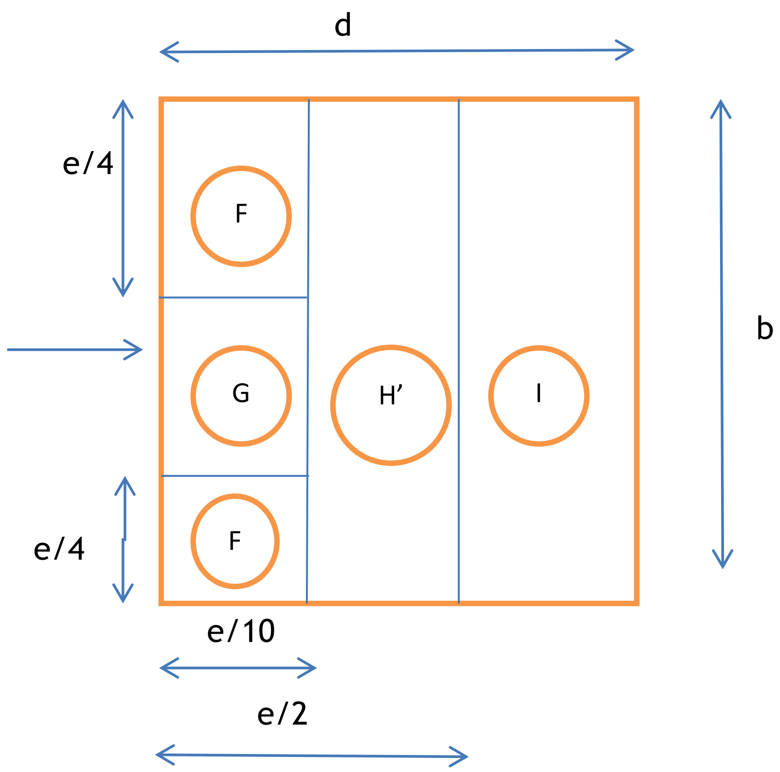


Figure D-9 Plan view of roof for wind in X-Direction

Areas: F = (4.8m) (12m) = 57.6m^2

G = (24m) (4.8m) = 115.2m^2

H' = (19.2m) (48m) = 921.6m^2

I = (18m) (48m) = 864m^2

Since $C_{pe} = C_{pe,10}$ for A ≥ 10m^2

At H = 75m, $W_e = q_{ref}\, C_e(Z_e) C_{pe}$, where q_{ref} = 0.23 and $C_e(Z_e)$ = 2.83

From Table D-2,

Zone	$C_{pe,10}$	We=(0.23)(2.83)(C_{pe})
F	-1.8	-1.172
G	-1.2	-0.781
H	-0.7	-0.456
I	+0.2 and -0.2	0.1302 and -0.1302

Elevation:-

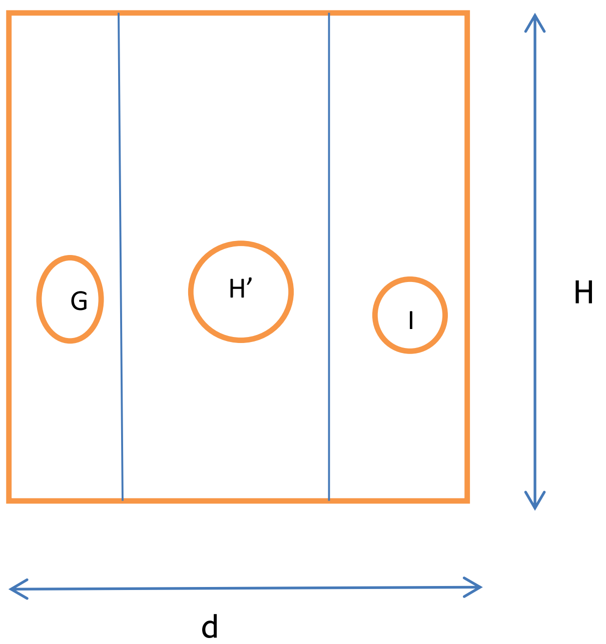


Figure D-10 Elevation view for wind in X-Direction

APPENDIX E

Analysis of shear walls for NSC and HSC

The parallel axis theorem is one of the methods used to derive second moment of area with respect to its centroidal axis *x'* or *y'*. When using this theorem, it is necessary to calculate the second moment of area with respect to a different parallel axis, say the *x* axis or *y* axis.

The parallel axis theorem states,

$$I_x = I_{x'} + A d_y^2$$

$$I_y = I_{y'} + A d_x^2$$

Where:

$I_{x'}$ second moment of area with respect to its centroidal axis *x'*

$I_{y'}$ second moment of area with respect to its centroidal axis *y'*

A Area of the shape

d_y Perpendicular distance between the *x'* and *x* axes

d_x Perpendicular distance between the *y'* and *y* axes

For both HSC and NSC structures that were analyzed, there are two U-type, two I-type and one H-type of shear walls. The NSC and HSC shear walls have thicknesses of 350mm and 300mm respectively. Below, as an example, analysis of shear walls for NSC model has been shown. The same procedure is used for the HSC model as well.

a) U-type of walls

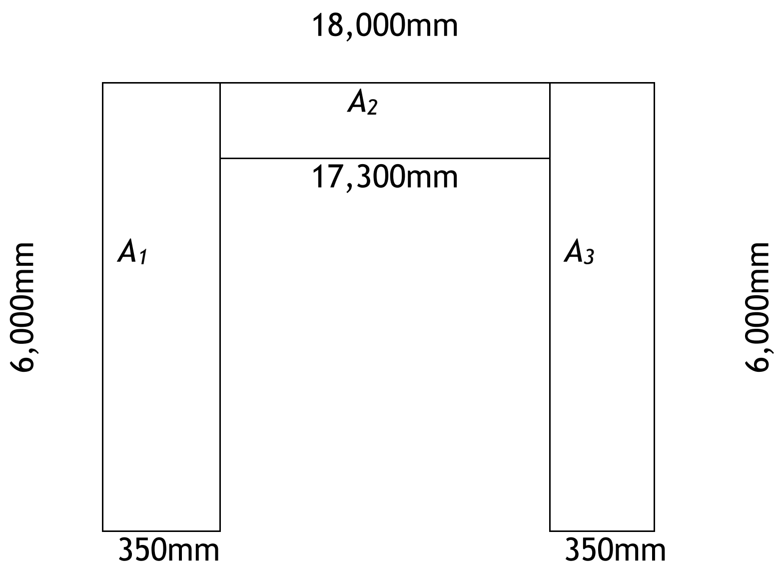


Figure E-1 U-type of wall with thickness of 350mm

Calculating the different areas of the U-type shear wall,

$A = (b)\ (h)$

$A_1 = (2.1)\ (10^6)\ mm^2$

$A_2 = (6.1)\ (10^6)\ mm^2$

$A_3 = (2.1)\ (10^6)\ mm^2$

Second moment of area with respect to its centroidal axis *x'*

$$I_{x'} = \frac{(b)\ (h^3)}{12}$$

$I_{1x'} = (6.3)\ (10^{12})\ mm^4$

$I_{2x'} = (6.2)\ (10^{10})\ mm^4$

$I_{3x'} = (6.3)\ (10^{12})\ mm^4$

Second moment of area with respect to its centroidal axis *y'*

$$I_{y'} = \frac{(h)\ (b^3)}{12}$$

$I_{1y'} = (2.12)\ (10^{10})\ mm^4$

$I_{2y'} = (1.5)\ (10^{14})\ mm^4$

$I_{3y'} = (2.2)\ (10^{10})\ mm^4$

Centroidal distance of areas with respect to the *x*-axis

$y_1 = 3{,}000mm$

$y_2 = 5{,}825mm$

$y_3 = 3{,}000mm$

Centroidal distance of areas with respect to the *y*-axis

$x_1 = 175mm$

$x_2 = 9{,}000mm$

$x_3 = 17{,}825mm$

$\Sigma A = A_1 + A_2 + A_3 = (10.3)\ (10^6)\ mm^2$

$A_1y_1 = (6.3)\ (10^9)\ mm^3$ $A_1x_1 = (3.7)\ (10^8)\ mm^3$

$A_2y_2 = (3.6)\ (10^{10})\ mm^3$ $A_2x_2 = (5.5)\ (10^{10})\ mm^3$

$A_3y_3 = (6.3)\ (10^9)\ mm^3$ $A_3x_3 = (3.7)\ (10^{10})\ mm^3$

Centroidal axis of the U-shaped shear wall can be calculated as follows,

$$Y = \frac{(A_1y_1)+(A_2y_2)+(A_3y_3)}{\Sigma A}$$

$Y = 4{,}718mm$

$$X = \frac{(A_1x_1)+(A_2x_2)+(A_3x_3)}{\Sigma A}$$

X = 9,029mm

The distance between the centroidal axis of the U-type of shear wall and the centroid of the areas is:

$d_1y = Y - y_1$ = 1,718mm

$d_2y = Y - y_2$ = 1,107mm

$d_3y = Y - y_3$ = 1,718mm

$d_1x = X - x_1$ = 8,854mm

$d_2x = X - x_2$ = 29mm

$d_3x = X - x_3$ = 8,796mm

The second moment of area with respect to the x-axis for the U-type of shear wall is calculated as follows

$$I_x = (I_{1x'} + A_1 d_1 y^2) + (I_{2x'} + A_2 d_2 y^2) + (I_{3x'} + A_3 d_3 y^2)$$

$$=(6.3)(10^{12}) + (2.1)(10^6)(1718)^2 + (6.2)(10^{10}) + (6.1)(10^6)(1107)^2 + (6.3)(10^{12}) + (2.1)(10^6)(1718)^2$$

$$= (1.3)(10^{13}) + (7.5)(10^{12}) + (1.3)(10^{13})$$

$$= (3.35)10^{13}\text{mm}^4$$

Since there are two U-type of walls, total second moment of area with respect to the x-axis is

$I_{x\ total}$= (3.35) (10^{13}mm^4) (2walls)

$I_{x\ total}$ = (6.7) (10^{13}) mm^4

The second moment of area with respect to the y-axis for the U-type of shear wall is calculated as follows

$$Iy = (I_{1y'} + A_1 d_1 x^2) + (I_{2y'} + A_2 d_2 x^2) + (I_{3y'} + A_3 d_3 x^2)$$

$$=(2.12)(10^{10}) + (2.1)(10^6)(8854)^2 + (1.5)(10^{14})+(6.1)(10^6)(29)^2 + (2.2)(10^{10}) + (2.1)(10^6)(8796)^2$$

$$= (1.7)(10^{14}) + (1.5)(10^{14}) + (1.7)(10^{14})$$

$$= (4.8)(10^{14})\ mm^4$$

Since there are two U-type of walls, total second moment of area with respect to the *y*-axis is

$$I_{y\ total} = (4.8)(10^{14} mm^4)(2walls)$$

$$I_{y\ total} = (9.6)(10^{14})mm^4$$

b) For H-type of wall

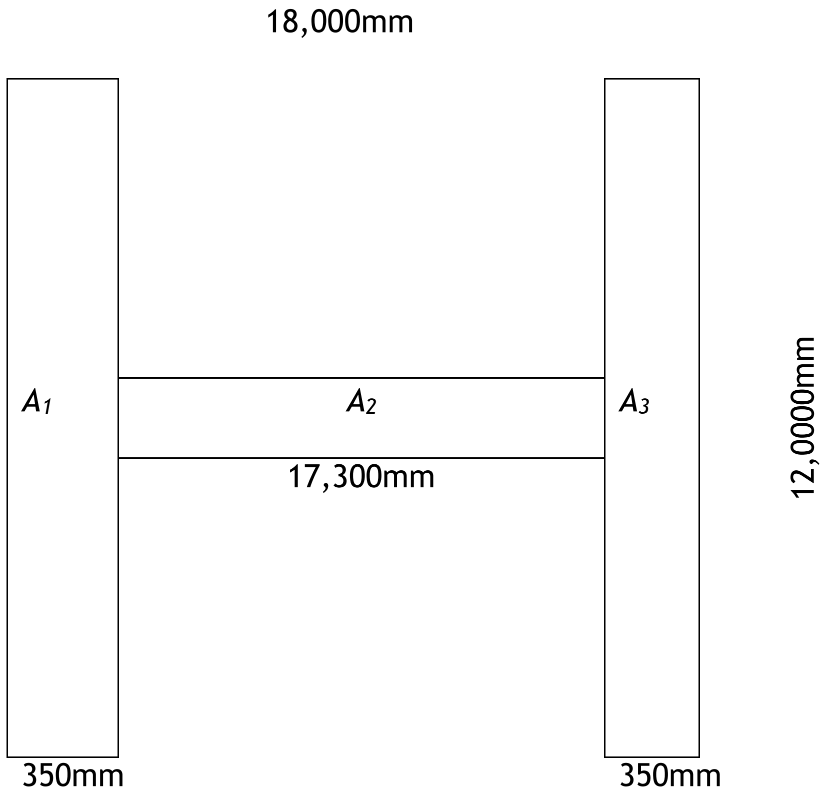


Figure E-2 H-Type of wall with thickness of 350mm

Calculating the different areas of the H-type shear wall,

$A = (b)\ (h)$

$A_1 = (4.2)\ (10^6)\ \text{mm}^2$

$A_2 = (6.1)\ (10^6)\ \text{mm}^2$

$A_3 = (4.2)\ (10^6)\ \text{mm}^2$

Second moment of area with respect to its centroidal axis *x'*

$$I_{x'} = \frac{(b)\ (h^3)}{12}$$

$I_{1x'} = (5.1)\ (10^{13})\ \text{mm}^4$

$I_{2x'} = (6.2)\ (10^{10})\ \text{mm}^4$

$I_{3x'} = (5.1)\ (10^{13})\ \text{mm}^4$

Second moment of area with respect to its centroidal axis *y'*

$$I_{y'} = \frac{(h)\ (b^3)}{12}$$

$I_{1y'} = (4.3)\ (10^{10})\ \text{mm}^4$

$I_{2y'} = (1.5)\ (10^{14})\ \text{mm}^4$

$I_{3y'} = (4.3)\ (10^{10})\ \text{mm}^4$

Centroidal distance of areas with respect to the *x*-axis

$y_1 = 6{,}000\text{mm}$

$y_2 = 6{,}000\text{mm}$

$y_3 = 6{,}000\text{mm}$

Centroidal distance of areas with respect to the *y*-axis

$x_1 = 175\text{mm}$

$x_2 = 9{,}000\text{mm}$

$x_3 = 17{,}825\text{mm}$

$\Sigma A = A_1 + A_2 + A_3$

ΣA= (14.5) (10^6) mm^2

A_1y_1 = (2.5) (10^{10}) mm^3 A_1x_1 = (7.4) ($10^8 mm^3$)

A_2y_2 = (3.7) (10^{10}) mm^3 A_2x_2 = (5.5) (10^{10}) mm^3

A_3y_3 = (2.5) (10^{10}) mm^3 A_3x_3 = (7.5) (10^{10}) mm^3

Centroidal axis of the H-shaped shear wall can be calculated as follows,

$$Y = \frac{(A_1y_1)+(A_2y_2)+(A_3y_3)}{\Sigma A}$$

Y = 6,000mm

$$X = \frac{(A_1x_1)+(A_2x_2)+(A_3x_3)}{\Sigma A}$$

X = 9,016mm

The distance between the centroidal axis of the H-type of shear wall and the centroid of the areas is:

$d_1y = Y - y_1 = 0$

$d_2y = Y - y_2 = 0$

$d_3y = Y - y_3 = 0$

$d_1x = X - x_1$ = 8,859mm

$d_2x = X - x_2$ = 34mm

$d_3x = X - x_3$ = 8,791mm

The second moment of area with respect to the *x*-axis for the H-type of shear wall is calculated as follows:

$$I_x = (I_{1x'} + A_1 d_1 y^2) + (I_{2x'} + A_2 d_2 y^2) + (I_{3x'} + A_3 d_3 y^2)$$

$= (5.1)\ (1013) + (6.2)\ (1010) + (5.1)\ (1013)$

$= (1.02)\ (10^{14})\ mm^4$

The second moment of area with respect to the *y*-axis for the H-type of shear wall is calculated as follows

$$Iy = (I_{1y'} + A_1 d_1 x^2) + (I_{2y'} + A_2 d_2 x^2) + (I_{3y'} + A_3 d_3 x^2)$$

$$= (4.3)(10^{10}) + (4.2)(10^6)(8859)^2 + (1.5)(10^{14}) + (6.1)(10^6)(34)^2 + (4.3)(10^{10}) + (4.2)(10^6)(8791)^2$$

$$= (8.5)\ (10^{14})\ mm^4$$

c) For I-type of walls

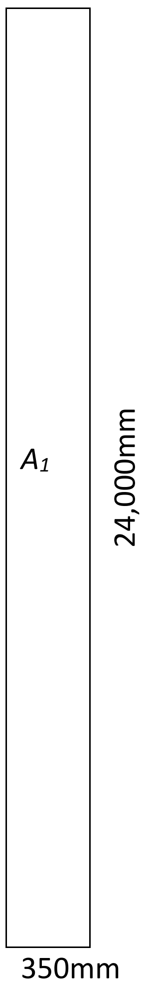


Figure E-3 I-type walls with thickness of 350mm

Calculating the area of the I-type shear wall,

$$A = (b)\ (h)$$

$$A_1 = (8.4)\ (10^6)\ \text{mm}^2$$

Second moment of area with respect to its centroidal axis *x'*

$$I_{x'} = \frac{(b)\ (h^3)}{12}$$

$$I_{1x'} = (4.03)\ (10^{14})\ \text{mm}^4$$

Second moment of area with respect to its centroidal axis *y'*

$$I_{y'} = \frac{(h)\ (b^3)}{12}$$

$$I_{1y'} = (8.5)\ (10^{10})\ \text{mm}^4$$

Centroidal distance of areas with respect to the *x*-axis

$$x_1 = 175\text{mm}$$

Centroidal distance of areas with respect to the *y*-axis

$$y_1 = 12{,}000\text{mm}$$

$$\Sigma A = A_1 + A_2 + A_3 \quad = \text{A}_1 = (8.4)\ (10^6)\ \text{mm}^2$$

Centroidal axis of the I-shaped shear wall can be calculated as follows,

$$Y = \frac{(A_1 y_1) + (A_2 y_2) + (A_3 y_3)}{\Sigma A}$$

$$Y = 12{,}000\text{mm}$$

$$X = \frac{(A_1x_1)+(A_2x_2)+(A_3x_3)}{\Sigma A}$$

X = 175mm

The distance between the centroidal axis of the I-type of shear wall and the centroid of the areas is:

$d_{1y} = Y - y_1 = 0 \qquad d_{x1} = X - x_1 = 0$

The second moment of area with respect to the *x*-axis for the I-type of shear wall is calculated as follows

$$I_x = (I_{1x'} + A_1 d_1 y^2)$$
$$= (4.03)\ (10^{14})\ mm^4$$

Since there are two I-type of walls, total second moment of area with respect to the *x*-axis is

$$I_{x\ total} = (4.03)\ (10^{14}\ mm^4)\ (2\ walls)$$
$$= (8.06)\ (10^{14} mm^4)$$

The second moment of area with respect to the *y*-axis for the I-type of shear wall is calculated as follows

$$I_y = (I_{1y'} + A_1 d_1 x^2)$$
$$= (8.5)\ (10^{10} mm^4)$$

Since there are two I-type of walls, total second moment of area with respect to the *y*-axis is

$$I_{y\ total} = (8.5)\ (10^{10} mm^4)\ (2\ walls)$$
$$= (1.7)\ (10^{11} mm^4)$$

The total second moment of area of the structure with respect to the *x*-axis is summed up as follows:

$\sum I_x = \sum I_x$ *of U-type walls* + $\sum I_x$ *of H-type wall* + $\sum I_x$ *of I-type walls*

$= (6.7)\ (10^{13}) + (1.02)\ (10^{14}) + (8.06)\ (10^{14})$

$= (9.75)\ (10^{14})\ \text{mm}^4$

The total second moment of area of the structure with respect to the *y*-axis is summed up as follows:

$\sum I_y = \sum I_y$ *of U-type walls* + $\sum I_y$ *of H-type wall* + $\sum I_y$ *of I-type walls*

$= (9.6)\ (10^{14}) + (8)\ (10^{14}) + (1.7)\ (10^{11})$

$= (1.76)\ (10^{15})\ \text{mm}^4$

According to Eurocode 2, global second order effects in buildings may be ignored if:

$$W \leq k_1 \frac{n_s}{n_s + 1.6} \frac{\sum E_{cd} I_c}{L^2}$$

Where:

W is the total vertical load of building

n_s is the number of stories

L the total height of building above level of moment restraint

E_{cd} the design value of the modulus of elasticity of concrete

I_c the second moment of area (uncracked concrete section) of bracing member(s)

k_1 relative flexibilities of rotational restraints where the recommended value is 0.31

For the Normal-Strength Concrete (NSC) model,

W = 1,053,235 KN = (10.05) (10^8) N

n_s = 25

L = 75 m = 75000 mm

E_{cd} = (31) (10^9) Pa = (31) (10^9) N/m^2=(31) (10^3) N/mm^2

ΣI_x = (9.75) (10^{14}) mm^4

ΣI_y = (1.76) (10^{15}) mm^4

K_1= 0.31

Case 1

$$(10.5)(10^8)\text{ N} \leq \frac{(0.31)\,(25)\,((31)\,(10^3\text{N/mm}^2))((9.75)(10^{14}\text{mm}^4))}{(25+1.6)\quad(75000\text{ mm})^2}$$

$$(10.5)(10^8)\text{ N} \leq (15.6)\,(10^8)\text{ N}$$

Global second order effects in the building may be ignored in the X direction.

Case 2

$$(10.5)(10^8)\text{ N} \leq \frac{(0.31)\,(25)((31)(10^3\text{N/mm}^2))((1.76)(10^{15}\text{mm}^4))}{(25+1.6)\quad(75000\text{ mm})^2}$$

$$(10.5)(10^8)\text{ N} \leq (28.9)\,(10^8)\text{ N}$$

Global second order effects in the building may be ignored in the Y direction. Thus, the overall global second order effects in the building may be ignored for the Normal-Strength Concrete (NSC).

Similarly, for High-Strength Concrete (HSC) structure, using 300mm wall thickness,

W = 1,002,737 KN = (10.03) (10^8) N

n_s = 25

L = 75 m = 75000 mm

E_{cd} = (42) (10^9) Pa = (42) (10^9) N/m^2 = (42) (10^3) N/mm^2

$\sum I_x$ = (8.3) (10^{14}) mm^4

$\sum I_y$ = (1.51) (10^{15}) mm^4

K_1= 0.31

Case 1

$$(10.03)(10^8)\ \text{N} \leq \frac{(0.31)\ (25)\ ((42)(10^3\text{N/mm}^2))((8.3)(10^{14}\text{mm}^4))}{(25+1.6)(75000\ \text{mm})^2}$$

$$(10.03)(10^8)\ \text{N} \leq (17.9)\ (10^8)\ \text{N}$$

Global second order effects in the building may be ignored in the X-direction.

Case 2

$$(10.03)(10^8)\ \text{N} \leq \frac{(0.31)\ (25)\ ((42)(10^3\text{N/mm}^2))((1.51)(10^{15}\text{mm}^4))}{(25+1.6)\ (75000\text{mm})^2}$$

$$(10.03)(10^8)\ \text{N} \leq (32.2)\ (10^8)\ \text{N}$$

Global second order effects in the building may be ignored in the Y-direction.

Thus, the overall global second order effects in the building may be ignored for the High-Strength Concrete (HSC) structure.

APPENDIX F

Seismic Analysis for NSC and HSC

- Building fundamental Period of vibration, T_1

$$T_1 = (C_t)\,(H^{3/4})$$

Where:

H is the height measured from the elevation of the specified bottom story/minimum level to the (top of the) specified top story/maximum level and is input in meters.

C_t = 0.085 for moment-resisting steel frames
= 0.075 for moment-resisting concrete frames
= 0.075 for eccentrically braced steel frames
= 0.05 for all other structures

$$T_1 = (0.075)\,(75m)^{3/4}$$
$$=1.912$$

- Design bedrock acceleration to acceleration of gravity, a

$$a = (a_o)\,(I)$$

Where:

a_o is the bedrock acceleration ratio for the site depending on the seismic zone

I is the importance factor

Since Addis Ababa is in zone II,

a_o =0.05

I=1 for ordinary buildings

$a = (a_o)\ (I) = (0.05)\ (1) = 0.05$

- Design response factor, β

$$\beta = (1.2S)/\ (T^{2/3})$$

Where:

S is the site coefficient

T is the fundamental period of vibration

For sub-soil class A, S=1

$$\beta = (1.2)\ (1)/\ (1.912)^{2/3}$$

$$= 0.738 < 2.5 \text{ Ok!}$$

- Behavior factor, γ

$$\gamma = \gamma_o\ k_D\ k_r\ k_w \leq 0.7$$

Where:

γ_o is the basic value of the behavior factor which depends upon the structural type

k_D is a factor for ductility class

k_r is a factor for regularity in elevation

k_w is a factor reflecting the prevailing failure mode in a structural system with walls

$\gamma_o = 0.2$

$k_D = 2.0$ for Low ductility, DCL (Low Energy Dissipative Structural Behavior)

$k_r = 1$ for regular buildings

$k_w = 1$ for frame and frame equivalent dual systems

$\gamma = \gamma_o\ k_D\ k_r\ k_w \leq 0.7$

$= (2.0)\ (1)\ (1)(0.2)$

$= 0.4 \leq 0.7$

- Ordinate of the design spectrum at period T_1, $S_d\,(T)$

$$S_d\,(T) = \alpha\ \beta\ \gamma$$

$$= (0.05)\ (0.738)\ (0.4)$$

$$= 0.0147$$

- Seismic base shear force for each main direction, F_b for Normal-Strength Concrete (NSC) model:

$$F_b = S_d\,(T_1)\ (W)$$

W = 1,053,235 kN

F_b = (0.0147) (1,053,235 kN)

=15,483 kN

$F_t = (0.07)\ (T_1)\ (F_b)$

F_b = concentrated force at the top

F_t = (0.07) (1.912) (15,483) kN

= 2,072.2 kN

- Distribution of the base shear for Normal-Strength concrete model:

$F_i = ((F_b - F_t)\ (W_i\ h_i)/\Sigma\ (W_i\ h_i)$

F_b-F_t = (15,483) – (2,072.2) kN

= 13,411kN

- Seismic base shear force for each main direction, F_b for High-Strength Concrete (HSC) model:

$$F_b = S_d\,(T_1)\,(W)$$

W = 1,002,737 kN

F_b = (0.0147) (1,002,737 kN)

= 14,740 kN

$F_t = (0.07)\,(Ti)\,(F_b)$

F_b = concentrated force at the top

F_t = (0.07) (1.912)(14,740) kN

=1,972.8 kN

- Distribution of the base shear for High-Strength Concrete

$F_i = ((F_b - F_t)\,(W_i\,h_i)/\sum\,(W_i\,h_i)$

$Fb - Ft$ = (14,740 - 1,972.8) kN

= 12,767.2 kN