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COMPARASION OF RUSSIAN STRUCTURAL CODE WITH IBC 2009 (US) STRUCTUREAL CODE IN TERMS OF SAFETY AND ECONOMY FOR AFGHANISTAN

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Abstract

Despite the practical use of International Building Code (IBC) code significantly increased by Afghan engineers during last decade, the Russians codes are still used and are familiar to many structural designers. Need for such documents to assist architects, engineers or code users to enable them transforming from Russian to IBC code were based for this thesis study. In addition, the intent was to indicate the implications of these codes on each other. The comparative analysis is undertaken in sequence based on IBC 2009, structural provisions Chapter 16. The sections of Chapter 16, IBC 2009 are listed sequentially with an analysis of comparison to the Snip 2-01-07.89.

Loads and Stresses Snip 2.01.07-89 1997 design requirements are different in many ways from the structural design provisions given in International Building Code (IBC) 2009, Chapter No.16. However, both codes mainly address the structural design requirements for dead, live, wind, snow, rain, and earthquake loads. As the result of first prototype building analysis comparisons, some differences were found; analysis results in terms of Moments based on IBC 2009 are higher, 30 to 40 percent than analysis results by SNIP 2.01.07-89. The effects mainly caused by the uniformly dead load UDL, safety factors and load combinations that are greater in IBC 2009 than proposed by Snip 2-01-07.89. The result of seismic base shear force calculated by IBC 2009 is smaller about 15 percent with result of seismic base shear force calculated accordance to Afghanistan Seismic Codes (1982-101-1 S / 1983-V-II SQN Afghanistan. The increase in seismic force by Afghanistan code caused by seismic coefficient difference and the building weight due to dead and live load differences. As the result of second prototype building design comparisons, some differences were also found. Design results in terms of flexure reinforcement area and column reinforcements based on IBC 2009 are higher about 50% and -16% than design results by SNIP 2.01.07-89 code for beam and column respectively. For same moment, column axial load and column uniaxial moment, IBC 2009 design reinforcement formulas will provide about 12% more reinforcement area for beam and 69% more reinforcement area for column respectively than calculated by SNIP 2.01.07-89.

Keywords: Comparison, Russian, Structural, IBC, Economy, Afghanistan

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INTRODUCTION

Though the devastating damage of the Afghanistan higher education system during the last three decades war following 1979 Soviet invasion on Afghanistan, the higher education system has not yet moved completely from Russian education system. Soviet Union influences on the region and its good academic relation with Afghanistan were two main reasons that most faculty and professionals did higher studies in Russia which resulted in the use of the Russian building code in Afghanistan.

Following new government establishment in 2001, the reconstruction and building works started in various disciplines and it was imposed by donors especially US Army Corps of Engineers to use International Building Codes in their funded projects. Despite the demand for adoption of International building code IBC, the Russian Codes are still used by most government ministries and departments such as Ministry of Urban Development Affairs MUDA, Municipalities, Ministry of Public Works, polytechnic university and others. No documents are available to assist architects, engineers or code users to enable them transforming from Russian codes to IBC codes and no study has been carried out to indicate the implications of these codes on each other in terms of safety and economy for specific requirements of Afghanistan. The problem is to investigate the comparative analysis of both codes with special consideration of Afghanistan local condition and applicability of codes.

COMPARATIVE ANALYSIS

Referenced document for comparative analysis is Chapter 16 of IBC 2009. Sections from Chapter 16 of IBC 2009 are listed in sequence with an analysis of comparison to the SNiP 2-01-07.89. The comparative analysis described each correlated sections' similarities and differences. But, not necessarily covered all differences and similarities that are in both codes' text and requirements. Instead the comparative analysis covered the important points that are necessary for architects, design engineers and code users. SNiP 2.01.07-89 1997 design requirements are different in many ways from the structural design provisions specified in International Building Code (IBC) 2009, Chapter No.16. But both codes mostly address the structural design requirements for dead, live, wind, snow, rain, and earthquake loads. For comparative analysis, chapter 16 of IBC 2009 is specifically compared with two different codes released by National Building Codes of Russia; a) Loads and stresses SNiP 2.01.07-89 1997 and b) Construction in seismic areas SNiP II-7-81 2001.

One significant difference is in the bases for determining the loads; in Loads and stresses SNiP 2.01.07-89 1997 loads are provided by the code itself but the IBC 2009 structural design requirements rely adopting portions for most of the loads from ASCE 7 Standard Minimum Design Loads for buildings and other structures.

Some Benefits of IBC 2009 this approach are explained as below;

1. The length of IBC 2009 code is reduced avoiding excessive details
2. Referenced portions by IBC 2009 are revised and updated by the ASCE through agreed process
3. For further useful information and commentary details, the code users can access to ASCE code.

Referenced section 1604 of IBC 2009 there is another significant difference of this section with SNiP; SNiP code only covers strength design requirements in two groups of ultimate limit states. The definition for strength design is mostly same with IBC 2009 but the limit states are defined in well details in Section 1.10 of SNiP 2-03-01.84. 1997.

Referenced section 1604.5 of IBC 2009 we can summarize some significant differences with SNiP code

1. First significant difference is in concept statement by both codes. IBC 2009 classifies building and other structures based on risk to human being, life, in the event of collapse or failure. But SNiP classifies building and other structures according to economic, sociological and ecological consequences of their collapse or failure.
2. Another significant difference is in the number of classified categories that IBC has four occupancy categories vs. three categories by SNiP code.

Referenced section 1605 of IBC 2009 we can summarize some significant differences with SNiP code

1. First significant difference in both codes is that IBC 2009 provides detail information about the limit state design methods of strength and allowable stress design. But, SNiP 2-01-07.89 1997 provides straight forward brief information about first limit state design method of strength design.

Table 1. Partial comparison between load combinations of IBC 2009 and SNiP codes using strength design method for limit state

IBC 2009 Fundamental Load Combinations in accordance Section 1605-2-1	SNiP 2-01-07.89 1997, Load Combinations	
1.2D + 1.6L ……Equation N0.16.2)	1.1D+1.2L (at full specified value 2 Kpa and more) 1.1D+1.3L (at full specified value less than	γ_{f1} x D x ψ_1 + γ_{f2} x L x ψ_2

	2 Kpa)	

Where: γ_{f1} = Reliability Coefficient provided in the Code, D = dead load L, Live load and ψ_1, ψ_2 are load combination coefficients

Based on comparison of section 1607-3 of IBC 2009 with SNiP code the average value of uniformly distributed live loads in IBC 2009 code for 39 corresponding occupancy categories are about 40% greater than the average value of uniformly distributed live loads values provided by SNiP code for same corresponding occupancy categories.

Referenced comparison of section 1607-5 of IBC 2009 with SNiP code: minimum requirement for partition loads of IBC section 1607-5 is 15 psf which is about 44% more than 10.4 psf (pound per square foot) proposed by SNiP coder for partition loads.

First Prototype building analysis:

A simple prototype building is considered for analysis. The main structural members such as slab, beam, and girder are discussed in terms of flexure analysis by both codes. It is intended to find out the differences in the result of analysis and its reasons caused these differences. In addition, the analysis will be extended to calculate seismic base shear forces for said building by both codes to enable us for clear comparative analysis of seismic force effects by both codes.

As the research intended to compare the structural provisions of International Building Code IBC 2009 with structural provisions of National Building Codes of Russia in terms of safety and economy considering the local condition of Afghanistan. One Story prototype building is considered to be located in Kabul, Capital of Afghanistan. In addition, the same analysis procedures are adopted to better understand the effects of both code provisions. Prototype building dimension and details are shown in Figure 1. The building dimension is 27 m x 20 m Restaurant Hall. A two brick wall is considered as parameter wall of the restaurant hall with thickness of 50 cm and main girder is provided running along 27 m direction and it is supported by two interior columns with 9m spacing. Beams are placed on girder directly and separating the girder by two 10 meters spans. The beams are spaced at 3 m c/c running along 20 m side of restaurant hall. Same analysis procedures are adopted to better understand the effects of both code provisions. For dead load consistency the main elements under analysis are sized same for both codes.

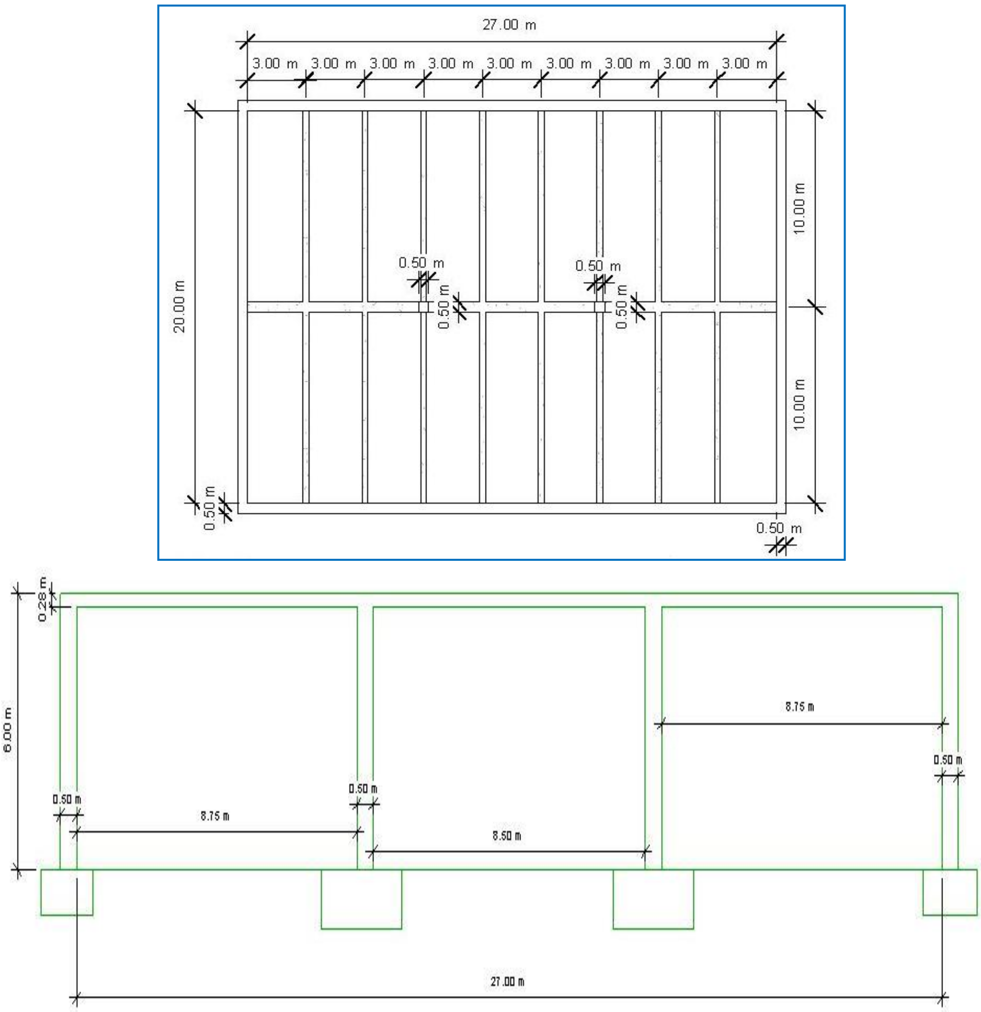


Figure 1. First Prototype Building Plan and Section

Building analysis by IBC 2009, ACI 318M-08

a) Slab Analysis: after necessary steps the slab thickness hf=15 cm and loads are summarized as below;

Service dead load (D.L) = 360+154+96 » 610 kg/m2

Service live load (L.L) 100 psf = 488.24 kg/m2 (for restaurant hall as per 1607.3 IBC 2009 table 1607.1)

Service Load (Ws) = 610 + 488.24 » 1098.24 kg/m2

Factored load (Wu) = 1.2 D + 1.6 L (Section 1605.2 IBC 2009)

Factored load (Wu) = 1.2 (610) + 1.6 (488.24)

Factored load (Wu) = 1513.184 kg/m2

As the slab is one way slab system and based on ACI 8-3-3 the clear spans of the slab are less than, 3 m (10 feet) and the external ends of slab are discontinue and not restrained. So, ACI moment coefficient analysis method applies

b) **Beam Analysis:** after necessary steps the beam sizes are h=60, b=30cm and loads are summarized as below;

Service dead load, D.L = 610 kg/m2

Service live load, L.L = 488.24 kg/m2

Beam sustaining 3 m slab. Thus, per meter running load shall be calculated as below;

Slab service dead load; = 610 x 3 = 1830 kg/m

Beam self weight, Service dead load; hw x bw x γ_c » 0.6 x 0.3 x 2400 = 432 kg/m

Total dead load =1830+432=2262 kg/m

Service live load = 488.24 kg/m2 x 3 m = 1464.72 kg/m

Ws = 2262 + 1464.72 = 3726.72 kg/m

Wu = 1.2xDL + 1.6x LL » 1.2x2262 +1.6x1464.72 $\Rightarrow$ Wu=5058 kg/m

Referenced ACI 8-3-.3 we use the ACI moment coefficient method for beam analysis;

c) **Girder/Beam Analysis:** Pointed loads on girder can be approximated from beams. Self weight of girder rib plus slab load directly resting on girder is considered as uniformly distribution loads on girder. after necessary steps the slab thickness h=100 cm and loads are summarized as below;

(U.D.L) self weight = 1.2 hwg x bwg x γ_c » 1.2 085 x 0.5 x 2400 = 1124 kg/m

Slab portion on Girder/Beam UDL, sg = wu (slab) x bwg » 1513.84 kg/m2 x 0.5m = 756.92 kg/m

Wg = UDL, Self weight + UDL, sg » 1124 kg/m + 756.92 kg/m Wg = 1980.92 kg/m

For analysis we use the moment distribution method

d) **Seismic Shear force calculation by IBC 2009:** Spectral acceleration values for Kabul, capital of Afghanistan as per IBC shall be calculated by USGS website; Location coordinates for Kabul is given below; Latitude 34.542° Longitude 69.135 °, For these coordinates the USGS website gives the below spectral acceleration values both for short period and one second period; Ss = 1.24 g, S1=0.5g

Site Class Determination;

As no soil investigation reports are available for the specific prototype building site. According to Section 1613.5.2 of IBC 2009 soil class D shall be applied.

Site coefficient determination;

Using the site class of D and the mapped acceleration parameters of 1.24 for short period and 0.5 for 1-second period , the following site coefficients can be retrieved; Fa=1.004 and Fv=1.5

Factors/parameters of adjusted maximum considered earthquake spectral response acceleration;

SMs = Ss x Fa = 1.24 x 1.004 = 1.245

SM1 = S1 x Fv = 0.5 x 1.5 = 0.75

Parameters/factors of design spectral response acceleration;

SDs = 2/3 x SMs = 2/3 x 1.245 = 0.83

SD1 = 2/3 x SM1 = 2/3 x 0.75 = 0.5

Category/Class of seismic Design;

As per table 1613-5-6, 1 and table 1613-5-6, 2 the seismic design category is;

Category/Class of seismic Design = D

Based on table 12-2-1 of ASCE we calculate response modification coefficients, R.

Moment resisting frame systems is special reinforced concrete moment frames. For this system the response modification coefficient is 8.

Seismic response Factors/coefficient;

As per section 12-8-1-1 ASCE the Cs is equal to;

$$C_s = \frac{SD_s}{(R/I)}$$

SDs = is the short period parameter of design spectral response acceleration.

R = response modification factor

I = Factor of importance

The value of Cs computed in accordance above equation shall not exceed

$$C_s = \frac{SD_1}{T(R/I)} \qquad \text{For } T \le TL$$

$$C_s = \frac{SD_1 x TL}{T^2(R/I)} \qquad \text{For } T > TL$$

Cs shall not be less than; Cs = 0.01

In addition, the building located where S1 is equal to or greater than 0.6 g, Cs shall not be less than

$$C_s = \frac{0.5 x S_1}{(R/I)}$$

So, $C_s = \frac{SD_s}{(R/I)}$

For SDs = 0.83, R=8

As per Table 1-1, ASCE the Occupancy category is III as under buildings/structures where more than 300 people come together in one area.

And as per table 11-5-1 importance factor for this Occupancy category is I=1.25

$$C_s = \frac{0.83}{(8/1.25)} = \frac{0.83}{6.4} = 0.1297$$

Seismic Base Shear (Section 12.8.1 ASCE);

V = Cs x W

W = Efficient seismic weight of the building as per section 12-7-2, ASCE

The Efficient seismic weight, w, of a building shall include the total load and 25% of the floor live load

Dead Loads; = 610 kg/m2 x (27x20) m2 » 610 x 540 = 329400 kg

25% Live load

= 25% x 488.24 x 540

= 65912.4 Kg

Total, w = 395,312.4 Kg

V = Cs x W

V = 0.1297 x 395,312.4 $\Rightarrow$ V = 51272.2 Kg = 512.722 KN

Building analysis by SNiP Code;

a) **Slab Analysis:** For load uniformity the slab thickness hf=15 cm and loads are summarized as below;

Total Normative (service) permanent loads; 360+154+96 » 610 kg/m^2

Total Design permanent loads; 396+200.2+124.8 » 721 kg/m^2

Temporary loads calculation;

- Long term loads = Nil
- Short term loads = accordance to table 3 section 3.5 of SNiP 2-0-07.89 1997 the normative load for restaurant hall is 3 Kpa.

As per section 3-7 of SNiP 2-0-07.89 1997 the reliability coefficients for uniformly distributed loads is 1.2 – at full specified value 2.0 kpa and more

So γ_f = 1.2 – at full specified value 2.0 kpa

Design live load = 3 kpa = 306 kg/m2 x 1.2 = 367.2 kg/m^2

As per section 1.12 SNiP 2-0-07.89 1997 in cause of main combinations including dead load and one live load (sustained or instantaneous) coefficients φ_1 and φ_2 shall not be considered. So, Design factored loads = 1.0 D + 1.0 L » 721 +367.2 = 1088.2 kg/m^2

As the slab is one way slab system and based on ACI 8-3-3 the clear spans of the slab are less than, 3 m (10 feet) and the external ends of slab are discontinue and not restrained. So, ACI moment coefficient analysis method applies.

b) **Beam Analysis:** For load uniformity the beam size is h=60, b=30cm cm and loads are summarized as below;

Total Design Permanent load (D.L) = 721 kg/m^2

Total design short term loads (L.L) = 367.2 kg/m^2

Beam is supporting 3m slab. Therefore, load per running meter will be as follows;

Design dead load from slab; = 721 kg/m^2 x 3m = 2163 kg/m^2

Normative dead load from beam self-weight = $h_w x b_w x \gamma_c$ = 0.6 x 0.3 x 2400 = 432 kg/m

Design dead load from beam self-weight; = 1.1 x 432 kg/m = 475.2 kg/m

Total design dead load = 2163 + 475.2 = 2638.2 kg/m

Total design live load = 367.2 kg/m2 x 3 = 1101.6 kg/m

Load combinations:

= 1.0 D.L + 1.0 L.L = 1.0x2638.2 +1.0 x 1101.6 = 3739.8 kg/m

Referenced ACI 8-3-.3 we use the ACI moment coefficient method for beam analysis;

c) **Girder/Beam Analysis:** Pointed loads on girder can be approximated from beams. Self weight of girder rib plus slab load directly resting on girder is considered as uniformly distribution loads on girder. after necessary steps the girder sizes are h=100cm, b=50cm and loads are summarized as below;

(i) P-is the point load on girder coming from beam due to factored loads;

P=2xWuxln/2 » 2x3739.8 x9.75/2 $\Rightarrow$ **P=36463.05 Kg**

(ii) Factored self-weight of the girder

= 1.1 ($h_{wg} x b_{wg} x \gamma_c$) = 1.1 x 0.85 x 2400 = 1122 kg/m

» 1513.84 kg/m2 x 0.5m = 756.92 kg/m

(iii) Part of slab on girder

(U.D.L) Sg = W_u (on slab) x b_{wg}

» 1088.2 kg/m^2 x 0.5m = 544.1 kg/m

W_g = (U.D.L) self weight + (U.D.L) S_g » 1122 kg/m + 544.1 kg/m

W_g = 1666.1 kg/m

For analysis we use the moment distribution method.

Shear force calculation according Afghanistan seismic codes (1982-101-1 S / 1983-V-II SQN:

As we described the seismic shear force calculation procedure according to SNiP code in section 1613.5.6, Determination of seismic design category of this research works. But, as the prototype building for the research is considered to be located in Kabul, capital of Afghanistan. Therefore, we need to calculate the shear force according to the Afghanistan seismic codes, developed by Ministry of public works, which is in turn based on Russian Codes considering the local condition for seismic parameter development. In addition, the SNiP code zoning maps not covered Kabul for getting the required seismic parameters. So, we have to use Afghanistan seismic code for building base shear force determination.

As per section 4-2-1-1 of 1982-101-1 S seismic shear force is calculated by the below formula;

$V_b = C \times \alpha_h \times w$

C – Coefficient describing the flexibility of building for increase of numbers of stories of the building

α_h – Seismic coefficient described accordance Section 3.4.1

w – Dead weight of the building + appropriate live load of the building

According, 1983-V-II SQN code building importance should be considered in above equations as per Section 3-1, Table no.4. According to above section the prototype building came under category no.1, residential, public and production buildings and structures and importance factor for this category is 1.0. So, no effect will have in the seismic result.

Now, we discussed each item in details;

α_h – as per section 3.4 Afghanistan is divided in 7-zones. Respect to the soil classes of the site, the seismic coefficients are provided for each zone.

Accordance Annexes H, Section 3.4.2 the seismic coefficient for Kabul city is;

$\alpha_h = 0.12$ Soil type 3 (Equaling to soil class-D)

$C = 9/n+5$,

n = number of stories but, C shall not be increased from 1.5 for frame structures and 1.33 for load bearing wall structures.

$n = 1$ » $C=9/1+5 = 9/6 = 1.5$ » So, $C = 1.33$

W = (dead weight of building) + (25% of live load)

Dead load = 610 kg/m^2 x (27x20) » 610 x 540 = 329,400 Kg

Dead load = 329,400 Kg

25 % of live load = 25% x 306 kg/m^2 x 540 m2

25 % of live load = 41,310 Kg

Total w = 370,710 Kg

$V_b = C \times \alpha_h \times w$ » 1.33 x 0.12 x 370,710 » **0.1596** x 370,710

V_b = 59,165.316 Kg, **V_b = 591.653 KN**

For similar W according IBC 2009 example, W=395,312.4 Kg. the Vb is

V_b = **0.1596** x 395,312.4 » V_b = 63091.86 kg, **Vb = 630.92 KN**

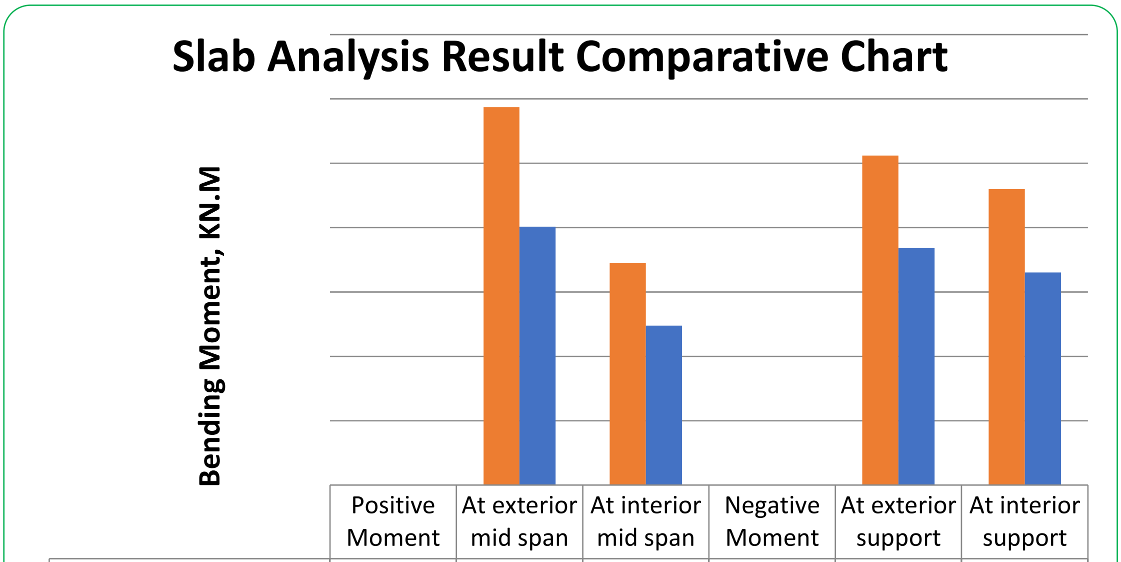
Comparison of the Analysis results accordance IBC 2009 and SNiP:

In this comparison the analysis results for structural elements of slab, beam, girder and horizontal base shear force are considered. Analysis calculation can be found in chapter 5. The analysis procedures for both codes are considered similar to best evaluate the code provisions for load consideration (normative and design), safety factors, load combinations and etc.

a) Analysis comparison for Slab system;

Table 2. Results of the slab analysis accordance to IBC and SNiP

Slab Moment Details	Analysis Result (KN.m) IBC	Analysis Result (KN.m) SNiP	How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP)%	Main reasons caused the differences
Positive Moment				IBC UDL load 488.24 kg/m2 SNiP is 305.92 kg/m^2 Load Combination; IBC; 1.2xDL+1.6x LL SNiP;1.1xDL+1.2x LL
At outer mid span	11.74	8.035	46%	
At inner mid span	6.895	4.958	39%	
Negative Moment				
At outer support	10.24	7.366	39%	
At inner support	9.193	6.611	39%	



	Positive Moment	At exterior mid span	At interior mid span	Negative Moment	At exterior support	At interior support
Analysis Result (KN.M) accordance to IBC		11.74	6.895		10.24	9.193
Analysis Result (KN.M) accordance to SNIP		8.035	4.958		7.366	6.611

Chart 1. Slab Analysis result comparative illustration accordance IBC and SNiP

b) Analysis comparison for Beam system;

Table 3. Results of the beam analysis accordance to IBC and SNiP

Beam Details	Analysis Result (KN.m) IBC	Analysis Result (KN.m) SNiP	How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP)%	Main reasons caused the differences
Moment at inner support				UDL loads; IBC 2009; 488.24 kg/m2 SNiP; 305.92 kg/m^2 **Load Combination;** IBC; 1.2xDL+1.6x LL SNiP;1.1xDL+1.2xL L
Negative Moment	534.25	395.02	35.25 %	
Positive moment at mid span	437.12	323.2	35.25 %	

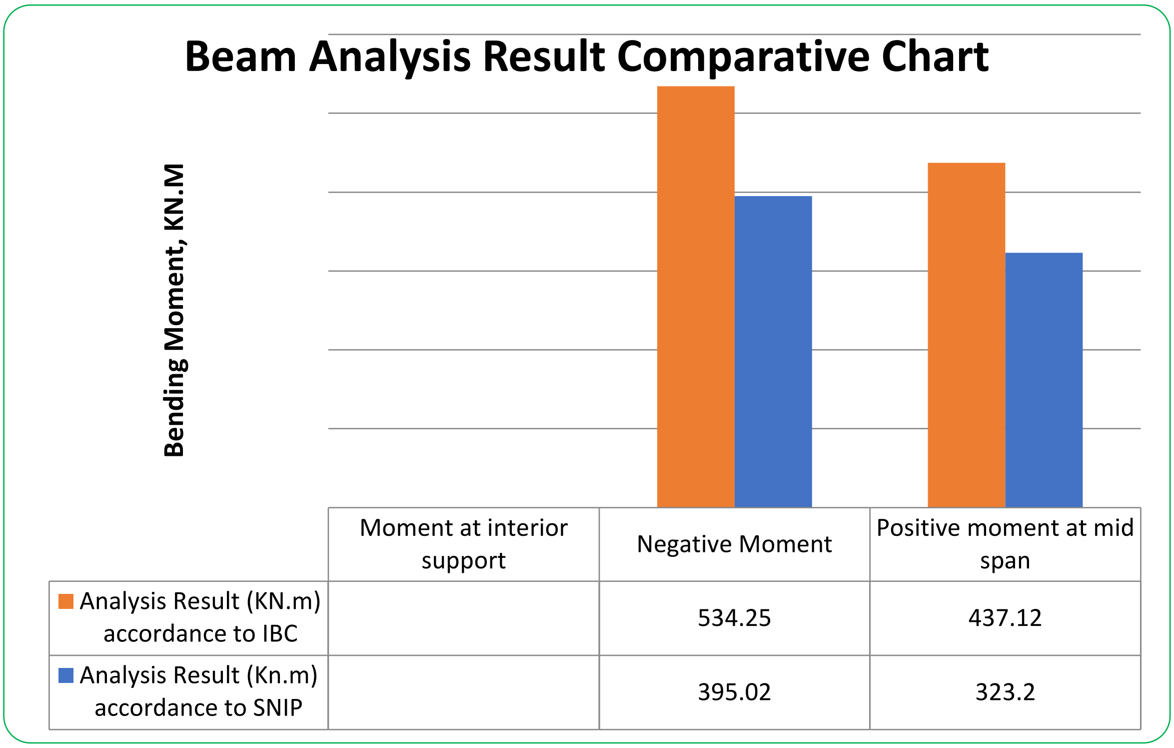


Chart 2. Beam Analysis result comparative illustration accordance IBC and SNiP

c) Analysis comparison for Girder system;

Table 4. Results of the Girder analysis accordance to IBC and SNIP

Girder	Analysis Result (KN.m) IBC	Analysis Result (KN.m) SNiP	How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP) %	Main reasons caused the differences
Final Moments				**UDL loads;** IBC 2009; 488.24 kg/m2 SNiP; 305.92 kg/m2 **Load Combination;** IBC; 1.2xDL+1.6x LL SNiP;1.1xDL+1.2xLL
FM_{ab}	0.00000	0.00000		
FM_{ba}	1342.334	1008.898	33.05%	
FM_{bc}	1342.334	1008.898	33.05%	
FM_{cb}	1342.373	1009.045	33.03%	
FM_{cd}	1342.373	1009.045	33.03%	
FM_{dc}	0.00000	0.00000		

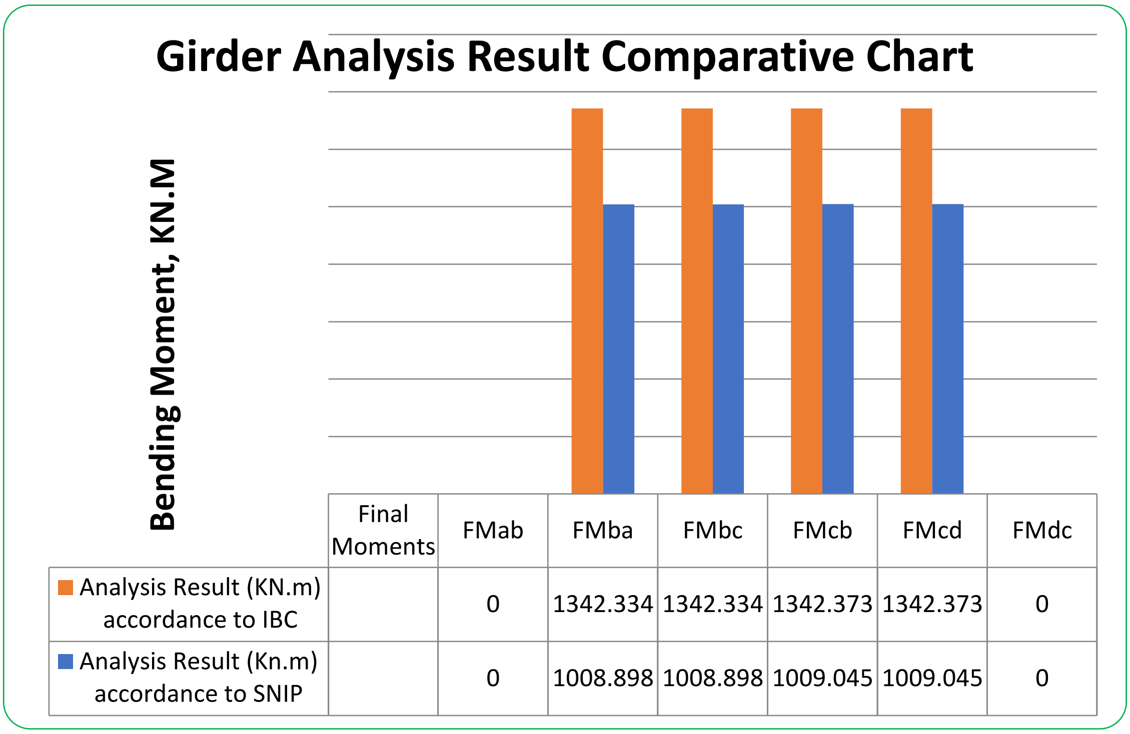


Chart 3. Beam Analysis result comparative illustration accordance IBC and SNiP

d) Analysis comparison for Seismic Base Shear;

Table 5. Results of the Seismic Shear Force analysis accordance to IBC and SNIP

Prototype Building	Seismic Base Shear (KN) accordance to IBC	Seismic Base Shear (KN) accordance to Afghanistan Seismic Codes (1982-101-1 S / 1983-V-II SQN:	How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP) %	Main reasons caused the differences
Seismic Base Shear for Original Building				**Building Weight;** IBC W; 395,312.4 Kg, SNiP;370,710 Kg **Importance Factor;** IBC; 1.25, SNiP;1 **Seismic Coefficient;** IBC=**0.1297**, SNiP=**0.1596**
V	**512.722**	**591.653**	**-13 %**	
Seismic Base Shear for same weight of w IBC=SNiP=395,312.4 Kg				**Building Weight;**

V	512.722	630.92	-19%	IBC W; 395,312.4 Kg, SNiP; 395,312.4 Kg **Importance Factor;** IBC; 1.25, SNiP;1 **Seismic Coefficient;** IBC=**0.1297,** SNiP=**0.1596**

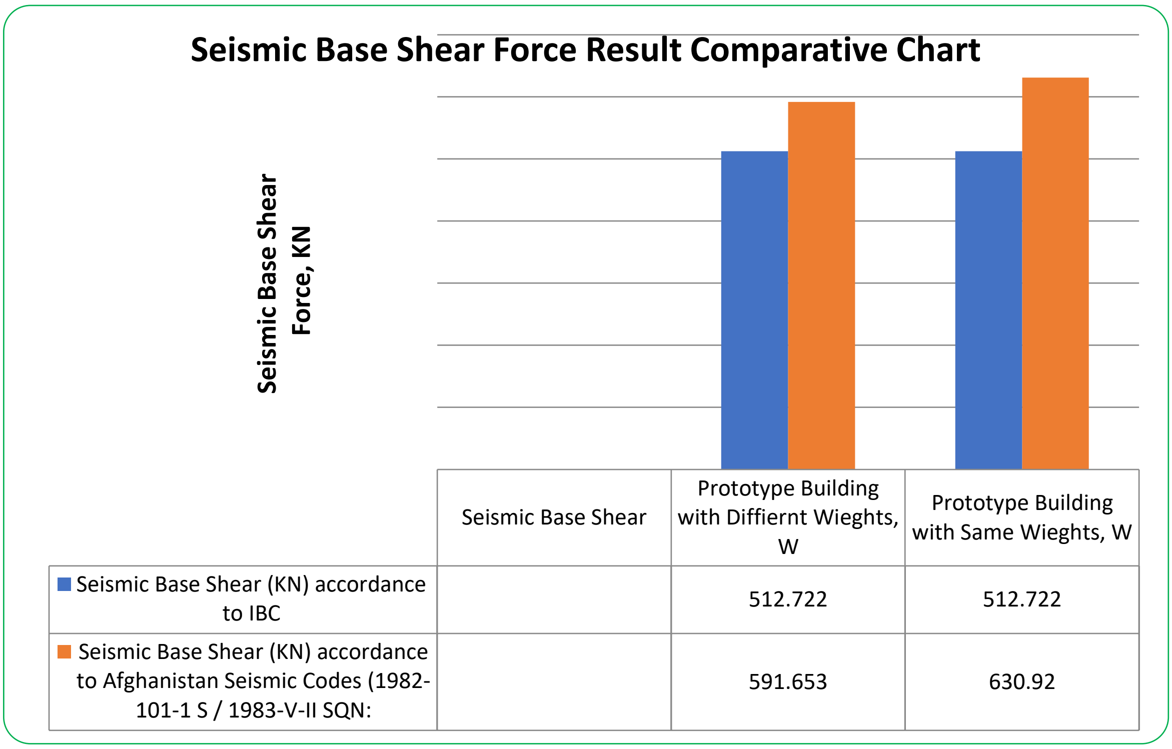


Chart 4. Seismic Base Shear Force Analysis result comparative illustration accordance IBC and Afghanistan Seismic Code based on SNiP

Second Prototype Building design

In order to find out the better implication of both codes on each other in respect of economy for condition of Afghanistan. The research supervisor proposed that a prototype building with frame Structure shall be designed for beam and column only.

The one story building is located in Kabul, Afghanistan with dimension of 6x4.6 m restaurant hall.

Design of Building by IBC 2009, ACI 318M-08

Material Properties;

Concrete compressive strength fc'	210	Kgf/cm2

Reinforcement strength fy			2,800 Kgf/cm2

Loads;

Live load (L.L) 100 psf	488.24	kg/m2	For restaurant hall as per 1607.3 IBC 2009 table 1607.1
Superimposed Dead load (S.D.L)	250.00	kg/m2	Mud thickness= 8cm and tiles thickness = 5 cm

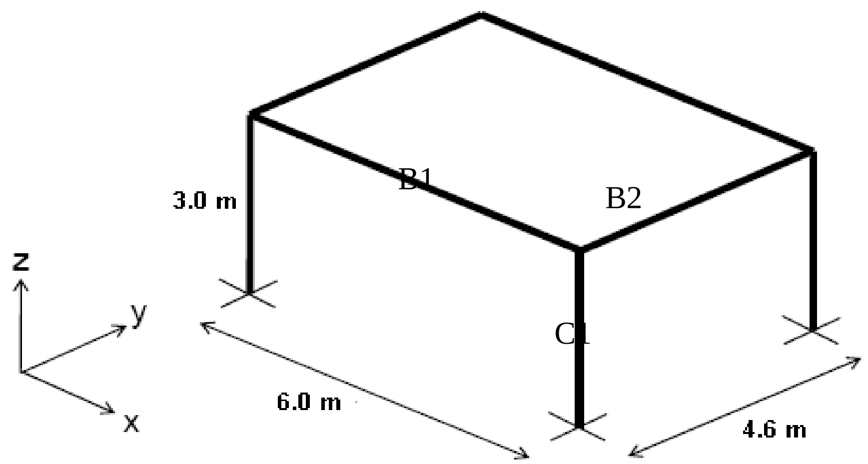


Figure 2. Second prototype building frame arrangements

Seismic Design Data:

Location coordinates for Kabul is given below;

Latitude 34.542°

Longitude 69.135°

For above coordinates the USGS website gives the below spectral acceleration values both for short and one second period;

Ss	1.24	g
S1	0.5	g
Soil class	D	

SIZES;

Slab

Assume hf	15	cm				

Beam

Assuming beam cross sectional dimension	(30x60)	cm	**bw**	30	**bh**	60
d=h-6	54	cm				

Column

Assuming column cross sectional dimension	(30x30)	cm	b	30	h	30

LOADS;

Gravity Loads;

Slab

Service dead load on slab	609.86	kg/m2	=(0.15*2400+0.08*1922+0.05*1922)
Service live load on slab	488.24	kg/m2	For restaurant hall as per 1607.3 IBC 2009 table 1607.1

Beam

Load on B1 and B2 are calculated in below table;

Table 6. Gravity load calculation for B1 & B2

Beam	Service dead load on slab (kg/m2)	Self weight of beam(kg/m)	Tributary Area (Ab), m2	LB (m)	Service dead load per meter (kg/m), DL*Ab/LB	Service Live Load on slab (kg/m2)	Service Live Load per Meter (Kg/m), LL*Ab/LB
B1	609.86	432.00	8.51	6.00	**864.98**	488.24	**692.49**
B2	609.86	432.00	5.29	4.60	**701.34**	488.24	**561.48**

Seismic Loads;

Site Coefficient determination;

Using the site class of D and the mapped acceleration parameters of 1.24 for short period and 0.5 for 1-second period

Fa	1.004
Fv	1.500

Adjusted maximum considered earthquake spectral response acceleration parameters;

SMs	Ss x Fa	1.2450
SM1	S1 x Fv	0.7500

Design spectral response acceleration parameters;

SDs	2/3 x SMs	0.83
SD1	2/3 x SM1	0.50

Seismic Design Category;

As per table 1613.5.6 (1) and table 1613.5.6 (2) the seismic design category is; D

Seismic response coefficients;

Based on table 12.2.1 of ASCE we are calculating response modification coefficients

Moment resisting frame system is special reinforced concrete moment frame for which the response modification factor (R) is equal to 8

As per section 12.8.1.1 ASCE the Cs is equal to

Cs = SDs / (R/I)

SDs	is the design spectral response acceleration parameter in the short period
R	response modification factor
I	the importance factor

The Value of Cs computed in accordance above equation shall not exceed

Cs = SD1 / T(R/I) for T≤TL

Cs = SD1*TL / T2(R/I) for T>TL

Cs shall not be less than;

Cs=0.01

In addition for structures located where S1 is equal to or greater than 0.6g, Cs shall not be less than;

Cs = 0.5*S1 / (R/I)

So, Cs = SDs / (R/I)

for SDs =	0.83	
R	8	
I=II	1	As per table 11.5-1 ASCE, Restaurant Hall with day-care facilities with capacity of more than 150
Cs	**0.1037**	=C98/ (C99/C100)

Seismic Base Shear (Section 12.8.1 ASCE);

V	**Cs * W**
W	The effective seismic weight as per section 12.7.2 ASCE. The effective seismic weight, W, of a structure shall include the total load and other loads as listed plus 25% of the floor live load

Total Dead load of the building

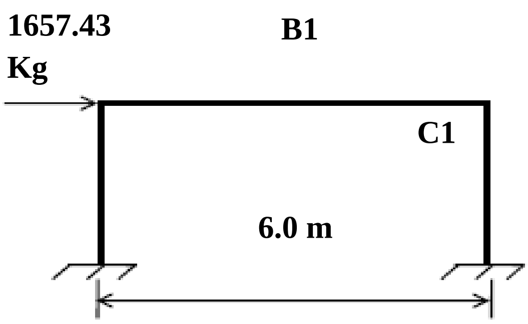
	Wslab+Wmud+Wtiles+Wbeams+Wcolumns			
	28,582.54	Kg		

25% live load

	3,368.86	Kg		
Total W	31,951.39	Kg		
V	Cs * W			
V	**3,314.85**	**Kg**	**48.76**	**KN**

The base shear shall be converted to story force in order to be analyzed. As the structure is single story the story forces assigned to story level in each direction is equal to = 4972.28 kg/2 =

1,657.43 **Kg**



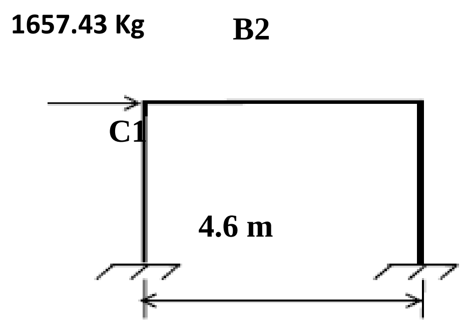


Figure 3. Shear Force application on beams

ANALYSIS;

Using E-TABS Nonlinear V9.0.4, the analysis is undertaken

Table 7. Analysis for flexure for Beams, Kg.M

Service load moments	B1			B2		
	-	+	-	-	+	-
DL	-1488.7	4680.4	-1328.3	-585.7	2381.4	-547.8
LL	-786.32	2471.5	-701.14	-286.1	1164.8	-268.6
EQL	-1184.7	0.0	-1144.1	1200.9	0.0	1167.8
Factored moments based on load combination; Section 1605.2.1 IBC 2009						
1.4D (Equation 16-1)	-2084.2	6552.5	-1859.6	-820.0	3334.0	-766.9
1.2D+1.6L (Equation 16-2)	-3044.6	9570.9	-2715.7	-1160.7	4721.4	-1087.2
1.2D+L (Equation 16-3)	-2572.8	8088.0	-2295.0	-989.0	4022.5	-926.0
1.2D+L (Equation 16-4)	-2572.8	8088.0	-2295.0	-989.0	4022.5	-926.0
1.2D+1.0 E+L (Equation 16-5)	-3757.5	8088.0	-3439.1	211.8	4022.5	241.8
1.2D-1.0 E+L	-1388.1	8088.0	-1151.0	-2189.9	4022.5	-2093.8
0.9D (Equation 16-6)	-1339.9	4212.3	-1195.4	-527.2	2143.3	-493.0
0.9D+1.0E (Equation 16-7)	-2524.6	4212.3	-	673.7	2143.3	674.8

			2339.5			
0.9D-1.0E	-155.2	4212.3	-51.4	-1728.0	2143.3	-1660.9
Maximum -ve	**-3757.5**		**-3439.1**	**-2189.9**		**-2093.8**
Maximum +ve		**9570.9**			**4721.4**	

Table 8. Analysis for flexure for Columns, Kg.M

Service load moments	**C1, Mx**			**C1, My**		
	Bottom	**Paxial**	**Top**	**Bottom**	**Paxial**	**Top**
DL	-609.6	4208.0	885.3	-247.4	4208.0	351.4
LL	-321.9	3368.9	467.7	-120.8	3368.9	171.4
EQL	1250.2	0.0	-681.6	1231.9	0.0	-652.6
Factored moments based on load combination; Section 1605.2.1 IBC 2009						
1.4D (Equation 16-1)	-853.4	5891.2	1239.4	-346.3	5891.2	491.9
1.2D+1.6L (Equation 16-2)	-1246.6	10439.8	1810.7	-490.2	10439.8	695.8
1.2D+L (Equation 16-3)	-1053.4	8418.5	1530.1	-417.7	8418.5	593.0
1.2D+L (Equation 16-4)	-1053.4	8418.5	1530.1	-417.7	8418.5	593.0
1.2D+1.0 E+L (Equation 16-5)	196.8	8418.5	848.5	814.2	8418.5	-59.6
1.2D-1.0 E+L	-2303.6	8418.5	2211.6	-1649.6	8418.5	1245.6
0.9D (Equation 16-6)	-548.6	3787.2	796.8	-222.7	3787.2	316.2
0.9D+1.0E (Equation 16-7)	701.6	3787.2	115.2	1009.2	3787.2	-336.3
0.9D-1.0E	-1798.8	3787.2	1478.3	-1454.5	3787.2	968.8
Maximum -ve	**-2303.6**		**115.2**	**-1649.6**		**-336.3**
Maximum +ve	**702**	**10,440**	**2,212**	**1,009**	**10,440**	**1,246**

Flexure Design of Beam

Step (a) According ACI 8.10, Beff for T-beam is minimum of;

i	16hf+bw	270	cm
ii	c/c span of beam / 4	142.5	cm
iii	c/c spacing between beams	600	cm

	So beff	142.5	cm			
Step (b)	Checking the beam if it is to be designed as rectangular beam or T-beam					
Trail #1						
	Assume a=hf	15	cm		Φ	0.9
	As=Mu / {Φfy (d-a/2)}	8.17	cm2			
	Recalculate a					
	a= As*fy/ (0.85 fc'*beff)	0.90	cm	<hf	So, design beam as rectangular beam	
Trail #2						
	As=Mu / {Φfy (d-a/2)}	7.09	cm2			
	a= As*fy/ (0.85 fc'*beff)	0.78	cm			
Trail #3						
	As=Mu / {Φfy (d-a/2)}	7.08	cm2			
	a= As*fy/ (0.85 fc'*beff)	0.78	cm			
	As	**7.08**	**Cm2**			
Step (c)	check for maximum and minimum reinforcement Asmax = ρ_{max}*bw*d					
	ρ_{max} = 0.85*β1(fc'/fy)*((εu/(εu+εy))			**β1**	0.85	0.
	ρ_{max}	0.020		**εu**		0.
					εy	
	Asmax = ρ_{max}*bw*d	32.92	cm2			

	Asmin = ρ_{min}*bw*d	8.10	cm2
	So As = Asmin =	**8.10**	**cm2**

Flexural Design of Column;

From analysis we have the following data for biaxial column

Factored load	**10,440**	**Kg**
Factored moment on X axis	2,304	Kg.m
Factored moment on Y axis	1649.6	Kg.m

Using approximate method to convert biaxial column moments to uniaxial moments

Mny / Mnx	0.72	<	b/h	1

So the following equation shall be used to convert biaxial case to uniaxial case of the column

Mnxo = Mnx + Mny (h/b)*(1-β)/β

where,

Mnx and Mny = Biaxial moment strengths

Mnxo = Uniaxial moment strengths

β	0.65				
Mnxo	**3192**	**Kg.m**			
b	30	cm	**h**	30	cm
f'c	210	kg/cm2			

Pu= Φ{0.85fc'ab+AsE*0.003(c-d')/c - AsE*0.003*(d-c/c)}

Trail #1

assume Ast = 1% of Ag	9.0	cm2
Φ	0.65	

d'	=4+1+2/2	6	cm	Using 20mm Main Steel, 10mm stirrups
d	h-d'	24	cm	

For finding the value of "C"

	10,458	So	**C**	7.14	cm

Pu=Φ*(0.85*fc'*β*c*b+As*MIN(fy;E*0.003*(c-d')/c))-As*MIN(fy;E*0.003*(d-c)/c)

As={Mu-Φ*0.425*fc'*a*b*(h-a)}/Φ[{(h/2)-d'}(fs1+fs2)]

fs1=0.003*E(c-d')/c	976.6	kg/cm2
fs2=0.003*E(d-c)/c	2,800.0	kg/cm2

With a=β1*c	6.069	cm	**β1=**	0.85
As 7.01	cm2			

After four trials the final As is found as below

As	5.64	cm2
Ast	11.27	cm2

Design of Building according SNIP 2.0.07-89 / SNIP II-7-81

Material Properties;

Concrete compressive strength fc'	210	kgf/cm2
Reinforcement strength fy	2,800	kgf/cm2

Loads;

Live load (L.L) 100 psf	488.24	kg/m2	Accordance to table 3 sections 3.5 of SNIP 2.01.07-89 the normative load for restaurant hall is 3 kpa or 305.92 kg/m2. But, for design comparison purposes we consider the load same as we did for IBC.

Superimposed Dead load (S.D.L)	250.00	kg/m2	Mud thickness= 8cm and tiles thickness = 5 cm

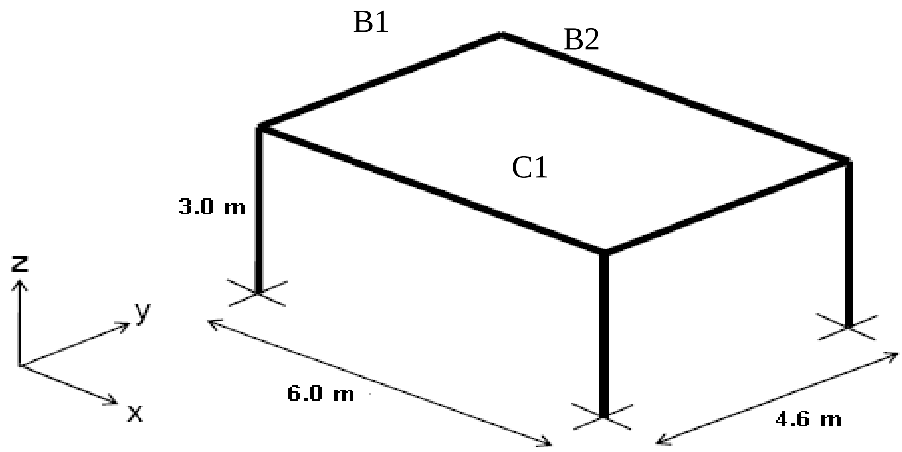


Figure 4. Second prototype building frame arrangements

Seismic Design Data:

As per section 3.4 Afghanistan Seismic codes (1982-101-1 S) Afghanistan is divided into seven seismic zones.

Kabul is located in Seismic zone No. VI with extremely high
risky area

According to Annexes H Section 3.4.2 the seismic coefficient for Kabul city is $\alpha h=0.12$ with respect to soil class of the sit.

αh	0.12	
Soil class	D	Soil Type 3 which is equal to Soil Class D

SIZES; same size as

LOADS;

Gravity Loads;

Slab

Service dead load (Normative Permanent loads)	609.86	kg/m2	=(0.15*2400+0.08*1922+0.05*1922)

Service live load on slab (Short term loads)	488.24	kg/m2	Accordance to table 3 sections 3.5 of SNIP 2.01.07-89 the normative load for restaurant hall is 3 kpa or 305.92 kg/m2. But, for design comparison purposes we consider the load same as we did for IBC.
Design service dead load (Design Permanent Loads)	720.818	kg/m2	=(0.15*2400*1.1+0.08*1922*1.3+0.05*1922*1.3) , According table 1, Section 2.2 SNIP 2.01.07-89 **The design loads = Normative load x Reliability coefficient**
Design Service live load (Design Live Load)	585.89	kg/m2	As per section 3.7. SNIP 2.01.07-89 the reliability coefficient for uniformly distributed loads is 1.2 - at full specified value 2.0 Kpa and more

Table 9. Gravity load calculation for B1 & B2;

Gravity load calculation for B1 & B2							
Beam	Design Service dead load on slab (kg/m2)	Self weight of beam (kg/m)	Tributary Area (Ab), m2	LB (m)	Design Service dead load per meter (kg/m), DL*Ab/LB	Design Service Live Load on slab (kg/m2)	Design Service Live Load per Meter (Kg/m), LL*Ab/LB
B1	720.82	432.00	8.51	6.00	**1022.36**	585.89	**830.98**
B2	720.82	432.00	5.29	4.60	**828.94**	585.89	**673.77**

Seismic Loads;

As per section 3.4 Afghanistan Seismic codes (1982-101-1 S) Afghanistan is divided by seven seismic zones.

Kabul is located in Seismic Zone No. VI with extremely high risky area

According Annexes H Section 3.4.2 the seismic coefficient for Kabul city is αh=0.12 with respect to soil class of the site

αh	0.12	
Soil class	D	Soil Type 3 which is equal to Soil Class D of IBC 2009

Site Coefficient determination;

As per section 4.2.1.1 of 1982-101-1 S seismic shear force is calculated by the below formula;

Vb = C x αh x w

C = coefficient describing the flexibility of building for increase the number of stories of the building

αh = Seismic coefficient described accordance Section 3.4.1

w = Dead weight of the building + appropriate live load of the building

αh	**0.12**	Soil Type 3 which is equal to Soil Class D

C = 9/n+5

n = number of stories 1

C shall not be exceeded from 1.5 for frame structures and 1.33 for load bearing wall structures

C **1.5**

W = (Dead weight of the building) + (25% of Normative Live load)

Total Dead load of the building		
	Wslab+Wmud+Wtiles+Wbeams+Wcolumns	
	28,582.54	Kg
25% live load		
	3,368.86	Kg
Total W	**31,951.39**	**Kg**

Vb	5,751.25	Kg	56.401	KN

The base shear shall be converted to story force in order to be analyzed. As the structure is single story, the story forces assigned to story level in each direction is equal to = 5751.25 kg/2 =

2,875.63 Kg

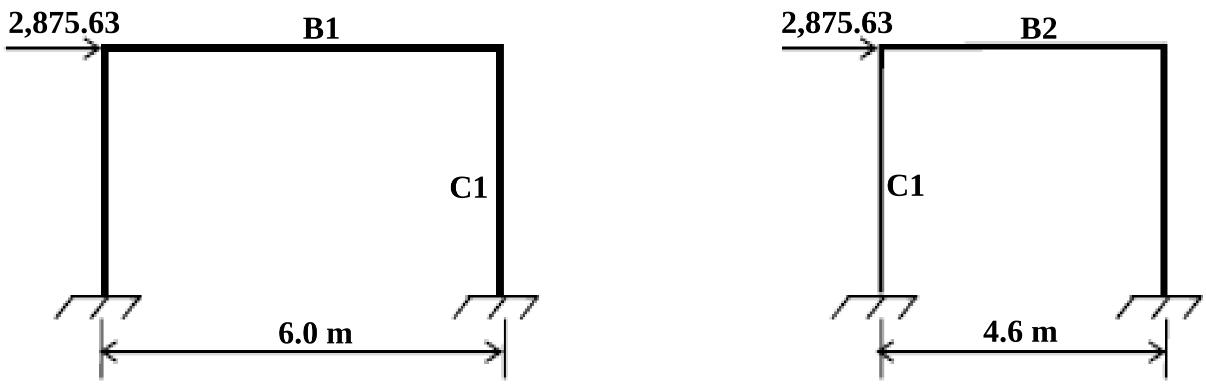


Figure 5. Shear Force application on beams

ANALYSIS;

Using E-TABS Nonlinear V9.0.4, for analysis

Table 10. Analysis for flexure for Beams, Kg.M

Service load moments	**B1**			**B2**		
	-	+	-	-	+	-
DL	-1667.4	5245.2	-1487.6	-650.8	2646.1	-608.9
LL	-943.57	2967.6	-841.36	-343.4	1397.7	-322.3
EQL	2062.1	0.0	-1977.6	2083.5	0.0	-2026.0

Factored moments based on load combination; , Section						
1.0 D+1.0 L	-2611.0	8212.8	-2329.0	-994.1	4043.9	-931.2
0.9 D+0.8 L + 1.0 E	-193.5	7094.7	-3989.5	1223.1	3499.7	-2831.9
0.9 D+0.8 L - 1.0 E	-4317.6	7094.7	-34.3	-2943.9	3499.7	1220.2
Maximum -ve	**-4317.6**		**-3989.5**	**-2943.9**		**-2831.9**
Maximum +ve	**-193.5**	**8212.8**			**4043.9**	

Table 11. Analysis for flexure for Columns, Kg.M

Service load moments	**C1, Mx**			**C1, My**		
	Bottom	**Paxial**	**Top**	**Bottom**	**Paxial**	**Top**
DL	-882.7	4208.0	991.6	-274.9	4208.0	390.3
LL	-386.3	3368.9	561.3	-145.0	3368.9	205.6
EQL	2145.7	0.0	-1135.9	2168.9	0.0	-1182.4
1.0 D+1.0 L	-1269.1	7576.9	1552.9	-419.8	7576.9	596.0
0.9 D+0.8 L + 1.0 E	1042.1	6482.3	205.6	1805.6	6482.3	-666.6
0.9 D+0.8 L - 1.0 E	-3249.2	6482.3	2477.4	-2532.3	6482.3	1698.2
Maximum -ve	**-3249.2**		**205.6**	**-2532.3**		**-666.6**
Maximum +ve	**1,042**	**7,577**	**2,477**	**1,806**	**7,577**	**1,698**

Flexure Design of Beam

Step (a)	According to addition to SNiP 2.03.01-84 Section 3.23, value of b'_f is calculated as below;		
i	Width of an overhang to each side of the rib must not be less than 1/6 of the span of the member	200	cm
ii	By cross ribs or hf '≥0.1h=6cm, 1/2 of clear	270	cm

distance b/w longitudinal ribs

So b'f **200** **cm**

Heavy-weight concrete B15 (equivalent to 3000 psi), for this the Rb = 7.7 Mpa or 78.5 kgf/cm2 according to table 8, with working condition γ_{b2}=0.9

Steel reinforcement Class A-II 280 Mpa (equivalent to 40,000 psi) , for this the Rs = 280 Mpa or 2850 kgf/cm2 according to table 15 (22, 23)

Rb	78.5	Kgf/cm2		
Rs	2850	Kgf/cm2		
Bending moment (M)	8212.8	Kg.m	821,275	kg.cm

Step (b) Calculation is made according to Item 3.22 with supposing that the compressed reinforcement is not required. So we check the condition (32) taking A's = 0

Rb*b'f*h'f(ho-0.5h'f)	10,950,750	Kg.cm	>	821,275

So the section should be design as rectangular section with the width b=b'f = 200 cm

b=b'f 200

Step (c) We determine the value α_m (coefficient) = M/Rb*b*ho2

α_m = M/Rb*b*ho2	0.0179	<	α_R=0.449	α_R according Table 18

So that the compressed reinforcement is not required

Step (d) The section area of tensile reinforcement is calculated by formula provided in Annex 2. For which according to Table 20 by α_m=0.0182 we find the value of ζ=0.990

As=M / (Rs*ζ*ho)	5.390	cm2

Check for Minimum steel requirement

Asmin=μ*b*h0/100		**μ=0.1 %**	0.1	For ζ=0.990 According Annex 2
Asmin	1.62	cm2	So Design steel is Ok	

Design of Column;

Reinforcement Area Calculation

From analysis we have the following data for biaxial column

Factored load	**7,577**	**Kg**
Factored moment on X axis	3,249	Kg.m
Factored moment on Y axis	2532	Kg.m

Using approximate method to convert biaxial column moments to uniaxial moments

Mny / Mnx	0.78	<	b/h	1

So the following equation shall be used to convert biaxial case to uniaxial case of the column

Mnxo = Mnx + Mny (h/b)*(1-β)/β

where,

Mnx and Mny = Biaxial moment strengths

Mnxo = Uniaxial moment strengths

β	0.65	
Mnxo	**4613**	**Kg.m**

Effective Length of the column

lo=1.2*3m=3.6m	3.6	m	[SNiP 2.03.01-84 Section 3.25] and [Guide to SNiP 2.03.01-84 Table 13]

Column Section

Width (b)	30	cm
Height (h)	30	cm

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

a	**5.00**	**cm**	[SNiP 2.03.01-84 Section 5.5]

Concrete Data

Initial modulus of elasticity during compression and tension			
Eb=235,000 Kg/cm2	235,000	Kg/cm2	Concrete Class B15 [SNiP 2.03.01-84 Table 29]
Design resistance of the concrete to axial compression for limiting states of the first group			
Rb=86.7 Kg/cm2	86.7	Kg/cm2	Concrete Class B15 [SNiP 2.03.01-84 Table 13]
Concrete specific conditions of use factor			
γb2=0.9	0.9		[SNiP 2.03.01-84 Table 15]

Reinforcement Data

Modulus of elasticity			
Es=2,100,000 Kg/cm2	2,100,000		Kg/cm2
Design resistance of the reinforcement to tension for limiting states of the first group			
Rs=2850 Kg/cm2	2850	Kg/cm2	A-II Class [SNiP 2.03.01-84 Table 22]
Longitudinal force due to the effects of continuous and long-term loads			
N1=7577 Kg	7,577	Kg	
Moment due to the effect of continuous and long-term loads			
M1=	4,613	Kg.m	
Longitudinal force due to the effects of continues and long-term loads			
Nsh=	0	kg	
Moment due to the effect of continues and long-term			

loads			
Msh=	0	kg.m	
Working Height of the section			
ho=h-a=30-5=25cm	25	cm	
Gyration radius of the section			
i_{min}=min(b,h)/√12	8.66	cm	
Slenderness			
λ_{max}=lo/i_{min}	41.57		
λ_{lim}=120			Limit Slenderness for columns, is 120 [SNiP 2.03.01-84 Section 5.3]
$\lambda_{max} \leq \lambda_{lim}$ = 1			
Bending Moment corresponding to the permanent, long-term and short-term loads except non long term loads such as wind and crane loads			
MI=M1+N1*Ho-a/2	80,382	Kg.m	
M=M1+Msh	4,613	Kg.m	
Total Longitudinal force			
Nt=N1+Nsh	7,577	Kg	
Bending moment corresponding to the all loads			
MII=M+Nt*(ho-a)/2	80,382	Kg.m	
γb1=min(0.9*MII/MI, 1.1)	0.90		
Rb=if(0.82*MII>MI, Rb*γb2, Rb*γb1)	78.03	kg/cm 2	[Guide to SNiP 2.03.01-84 Section 3.1]
Io/h < 14	12	So the effect shall not be	

		considered	
			Second Order Effects [SNiP 2.03.01-84 Section 3.24]
Factor assumed depending on concrete type			
$\beta=1$	1	for heavy weight concrete [Guide to SNiP 2.03.01-84 Table 30	

Factor accounting for the effect of a long duration load of member limit state bend which shall not be more than $1+\beta$

$\Phi 1=\min(1+\beta^*MI/MII, 1+\beta)$	2.00	[SNiP 2.03.01-84 Section 3.6 (21)]	
Random eccentricity of of longitudinal force			
ea=max(h/30,lo/600)	1.00	cm	[SNiP 2.03.01-84 Section 1.21]
Eccentricity of longitudinal force			
e0=(M/Nt)+ea	61.88	cm	
δe is coefficient assumed to be e0/h but not less than $\delta e.min$			
$\delta e.min=0.5-0.01^*I0/h-0.01^*Rb$	-0.40		[SNiP 2.03.01-84 Section 3.6 (22)]
$\delta e=\max(e0/h, \delta e.min)$	2.06		[Guide to SNiP 2.03.01-84 Section 3.54]
Assume the Reinforcing ratio $\mu=0.001$	0.001		
$\alpha=Es/Eb$	8.94		
Conditional Critical force			[Guide to SNiP 2.03.01-84 Section 3.54 (93), SNiP 2.03.01-84 Section 3.6 (20)]

$Ncr=1.6^*Eb^*b^*h/(l0/h)2[(0.11/(0.1+\delta e)+0.1)/3\Phi 1+\mu^*\alpha(h0-a/h)2]$

Ncr=	245,687	Kg

The coefficient η taking into account the effects of deflection on the eccentricity of longitudinal force **e0** shall be found from

η=1/(1-Nt/Ncr)	1.0318	[SNiP 2.03.01-84 Section 3.6 (19)]

Distance from the point of application of longitudinal force N to the resultant of the forces in reinforcement

e=e0*η+h0-a/2	73.85	cm

Relative value of longitudinal force

αn=Nt/Rb*b*h0	0.129	[Guide to SNiP 2.03.01-84 Section 3.62]

Relative value of moment

αm1=Nt*e/Rb*b*h0^2	0.3824	[Guide to SNiP 2.03.01-84 Section 3.62]
δ=a/h0	0.200	[Guide to SNiP 2.03.01-84 Section 3.62]

Coefficient taken for the concrete types

α=0.85	0.85	for normal weight concrete [SNiP 2.03.01-84 Section 3.12]

Characteristic of the compression zone of concrete is equal to

ω=α-0.008*Rb/1kg/cm2	0.226	[SNiP 2.03.01-84 Section 3.12 (26)]

The ultimate stress in reinforcement of compressed zone, taken for structures from heavy-weight concrete in terms of design loads

5000 kg/cm2 for γb2=0.9 otherwise 4000 kg/cm2

as γb2=0.9 so σsc.u=5000 kg/cm2	5000	kg/cm2

Limit value pf relative height of the compressed zone

$\xi R = \omega/(1+Rs/\sigma sc.u*(1-\omega/1.1)$	0.155	[SNiP 2.03.01-84 Section 3.12 (25)]
$\alpha s = \alpha m1-\alpha n*(1-\alpha n/2)/1-\delta$	0.3267	[Guide to SNiP 2.03.01-84 Section 3.62 (114)]

Relative height of the compressed zone

$\xi=(\alpha n*(1-\xi R)+2*\alpha s*\xi R)/1-\xi R+2*\alpha s$	0.1408	[Guide to SNiP 2.03.01-84 Section 3.62 (109)]

Area of compressed or tensile reinforcement (for symmetric arrangement)

$As=if[\alpha n<\xi R, Rb*b*h0/Rs*\alpha m1-\alpha n*(1-\alpha n/2)/1-\delta, Rb*b*h0/Rs*\alpha m1-\xi*(1-\xi/2)/1-\delta]$

[Guide to SNiP 2.03.01-84 Section 3.62 (112 and 113)]

as $\alpha n<\xi R$, So $As =(Rb*b*h0/Rs)*(\alpha m1-(\alpha n*(1-\alpha n/2)))/1-\delta$

	6.7084	cm2
Actual reinforcement ratio		
$\mu_{actual}=As*2/b*h0$	0.018	
Gyration radius i	8.66	
Slenderness ratio λ	41.57	
Minimal Reinforcing ratio		[Guide to SNiP 2.03.01-84 Section 3.62 (112 and 113)]

$\mu_{minimul}=if(\lambda<17,0.1, if(\lambda\geq17\wedge\lambda\leq35,0.2, if(\lambda>35\lambda\leq83,0.4,0.5)))/100$

$\mu_{minimul}$ =	0.004	[Guide to SNiP 2.03.01-84 Table 47]

Total Reinforcement Area shall not be less than the minimum value

$A_{totol}=if(\mu_{actual}<\mu_{minimul}, \mu_{minimul}*b*h0, 2*As)$

Astotal	**13.42**	**cm2**

Design Result Comparison:

Table 12. Results of the Beam and Column Design according to IBC and SNiP

<table>
<tr><th>Prototype Building</th><th>Design of Building by IBC 2009, ACI 318M-08</th><th>Design of Building according SNIP 2.0.07-89 / SNIP II-7-81</th><th>How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP) %</th><th>Main reasons caused the differences</th></tr>
<tr><td colspan="4" rowspan="20">Steel Reinforcement in Cm2 for Original Building Analysis Moments</td><td>Building Weight;</td></tr>
<tr><td>IBC W; 395,312.4 Kg, SNiP; 395,312.4 Kg</td></tr>
<tr><td>Importance Factor;</td></tr>
<tr><td>IBC; 1, SNiP;1</td></tr>
<tr><td>Seismic Coefficient;</td></tr>
<tr><td>IBC=0.104 SNiP=0.180</td></tr>
<tr><td>Load Combination:</td></tr>
<tr><td>IBC = 1.2D+1.6L
SNiP = 1.0 D+1.0 L</td></tr>
<tr><td>Beam Moment, Kg.m</td></tr>
<tr><td>IBC = 9570.90
SNiP = 8212.80</td></tr>
<tr><td>Column Uniaxial Moment, Kg.m</td></tr>
<tr><td>IBC = 3191.85
SNiP = 4612.71</td></tr>
<tr><td>Column Axial Load, Kg</td></tr>
<tr><td>IBC = 10439.85
SNiP = 7576.89</td></tr>
<tr><td>a, cm</td></tr>
<tr><td>a= As*fy/ (0.85 fc'*beff) = 0.78cm</td></tr>
<tr><td>αm = M/Rb*b*ho2</td></tr>
</table>

				As, cm2
Beam	**8.10**	**5.39**	50.27%	As=Mu / {Φfy (d-a/2)} Þ Mu / fy * 48.25
Column	**11.27**	**13.42**	-15.97%	As=M / (Rs*ζ*ho) Þ M / Rs * 53.46
Steel Reinforcement in Cm2 for same Moments				**Beam Moment, Kg.m**
				IBC = 20,000 SNiP = 20,000
				Column Uniaxial Moment, Kg.m
				IBC = 5000 SNiP = 5000
Beam	**14.92**	**13.33**	11.97%	**Column Axial Load, Kg**
Column	**24.51**	**14.48**	69.24%	IBC = 15000 SNiP = 15000

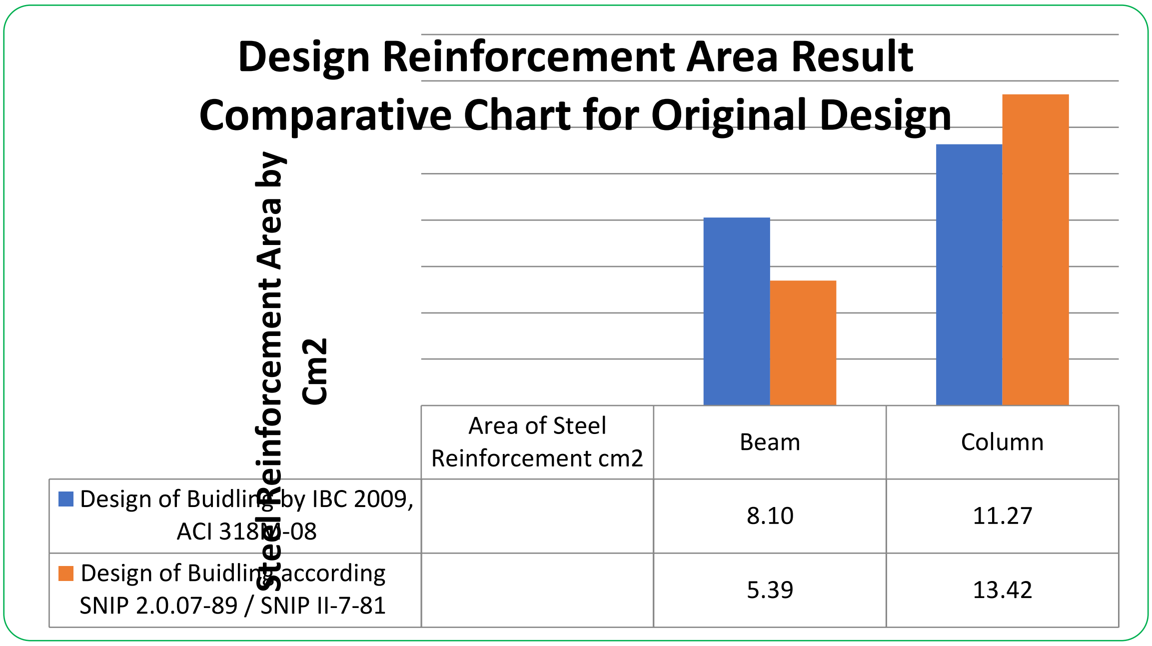
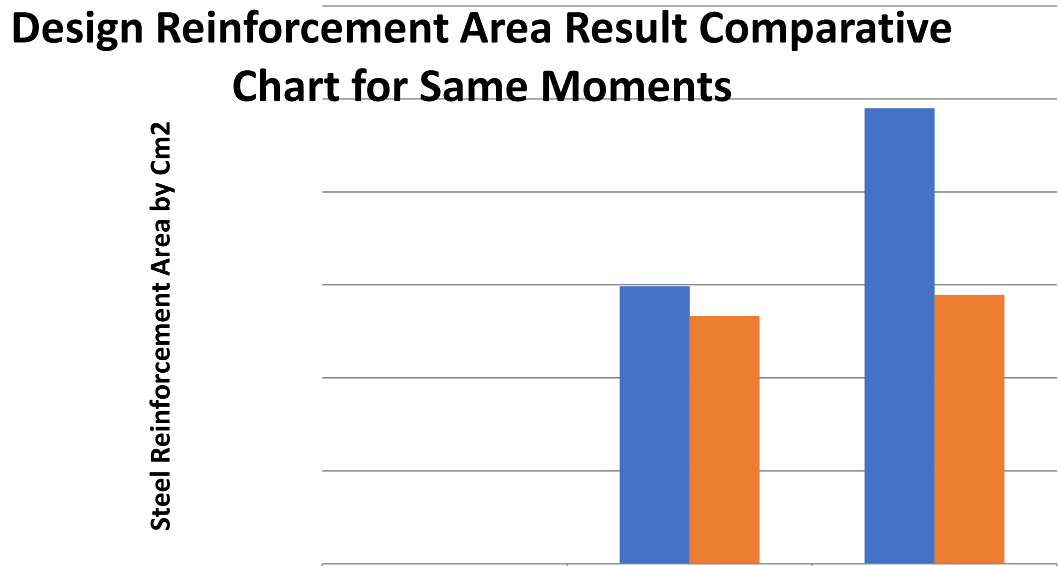


Chart 5. Design result comparative illustration according IBC and SNiP



	Area of Steel Reinforcement cm2	Beam	Column
Design of Buidling by IBC 2009, ACI 318M-08		14.92	24.51
Design of Buidling according SNIP 2.0.07-89 / SNIP II-7-81		13.33	14.48

Chart 6. Design result comparative illustration according IBC and SNiP

Conclusions;

Chapter 16, structural provisions of International Building Code IBC 2009 design requirements are different in many ways from the structural design provisions provided in SNiP 2.01.07-89 1997 as discussed in comparative analysis. However, both codes provide the structural requirements for dead, live, wind, snow, rain and earthquake loads.

- First significant difference is in the bases for determining the loads in IBC 2009 which generally rely adopting portions for most of the loads from ASCE 7. Instead, the SNiP code determining the loads by the code itself.
- Other significant difference is in the concept based on which the building and structures are classified under various occupancy categories. IBC 2009 classifying building occupancies based on hazard to human life in the event of failure, instead the SNiP classifying building and other structures based on economic, sociological and ecological consequences of their failure. Numbers of classified building occupancy categories are also different in both codes.
- Other main significant difference in both codes is in the application of safety factors and the load combinations. This difference really affects the load calculation dissimilarities accordance both codes. For example, basic load combination of dead and live load according to IBC 2009 section 1605.2.1 is;

1.2 D + 1.6 L instead the main combination expressed for similar loads is; 1.1 D + 1.2 L (at full specified value 2.0 Kpa and more) and 1.1D + 1.3 L (at full specified value less than 2.0 Kpa).

- In loading section, other significant difference of both codes is in values of uniformly distributed live loads. For most causes the uniformly distributed live load values provided in IBC 2009, section 1607.3 are greater than the uniformly distributed live load values provided in table 3 of SNiP 2.01.07-89 1997.
- Minimum requirements for partition loads accordance to section 1607.5 of IBC 2009 is about 44% more than proposed by the Section 3.6 of SNiP 2.01.07-89 1997
- For similar occupancy with same structural requirements there is 30% percent more reduction in live load according to SNiP 2.01.07-89 1997 vs. the IBC 2009 code.
- Criteria for flat and sloped roof are different in both codes. According to the Section 7.3 of a ASCE a roof with a slope less than or equal to 5 degrees shall be calculated by (7-1) formula as flat roof with consideration of exposure, thermal and importance factors. But, according to SNiP 2.01.07-89 1997, appendix 3 a roof with slope less than or equal to 25 degrees shall be calculated as flat roof.
- The slope degree for a roof to be qualify as free of snow load is more than 70 degrees accordance IBC 2009 and more than or equal to 60 degrees accordance to SNiP 2.01.07-89 1997.
- Based on the analysis of first prototype building and design of second prototype building by both codes; SNiP 2.01.07-89 1997 code is economic vs. IBC 2009 code.
- Considering the last decade developments in the field of higher education and opportunity for greater use of IBC code, it is recommended for engineers and code users to adopt the IBC code in building structural design.

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