

**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 percents 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: IBC 2009, SNiP Code, Structural Code Comparisons, Russian Building Codes

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 + 1.6LEquation N0.16.2)	1.1D+1.2L (at full specified value 2 Kpa and more)	$\gamma_{D1} \times D \times \psi_1 + \gamma_{L2}$
	1.1D+1.3L (at full specified value less than 2 Kpa)	$\times L \times \psi_2$

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.

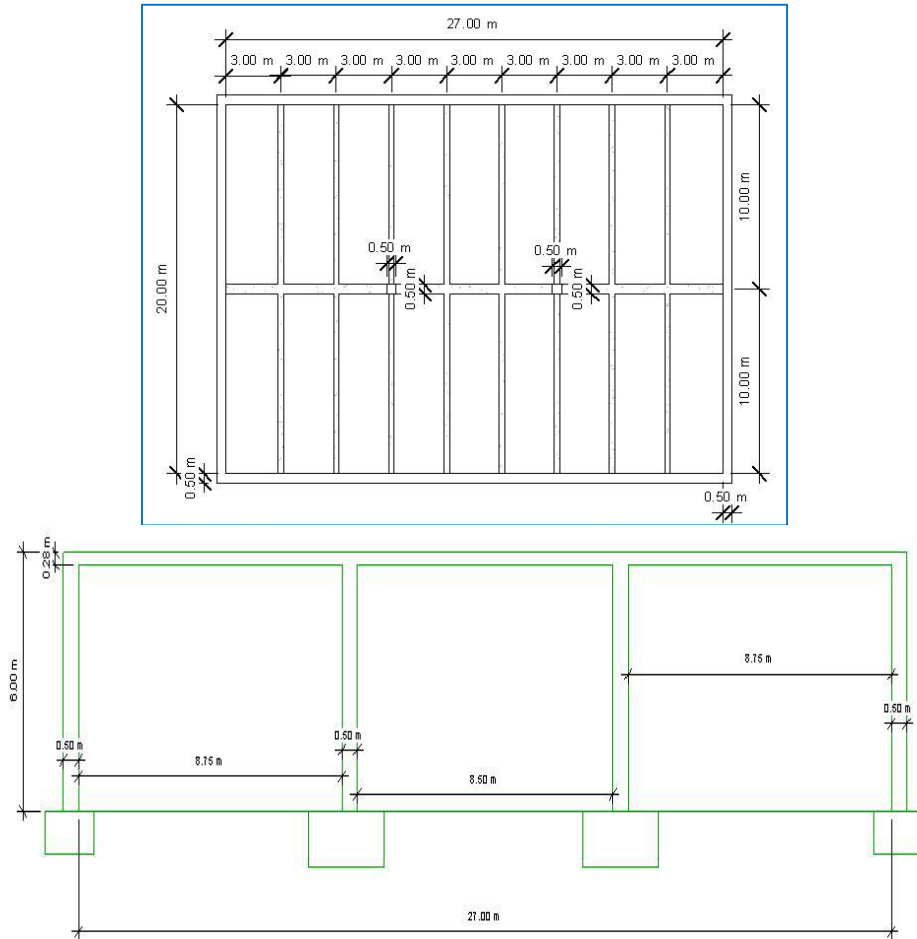


Figure 1. First Prototype Building Plan and Section

Building analysis by IBC 2009, ACI 318M-08

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

Service dead load (D.L) = $360+154+96 \gg 610$ 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) = $610 + 488.24 \gg 1098.24$ 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 (W_u) = 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 discontinuous and not restrained. So, ACI moment coefficient analysis method applies

b) Beam Analysis: after necessary steps the beam sizes are $h=60$, $b=30$ cm 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; = $610 \times 3 = 1830$ kg/m

Beam self weight, Service dead load; $hw \times bw \times \gamma_c \gg 0.6 \times 0.3 \times 2400 = 432$ kg/m

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

Service live load = $488.24 \text{ kg/m}^2 \times 3 \text{ m} = 1464.72$ kg/m

$W_s = 2262 + 1464.72 = 3726.72$ kg/m

$W_u = 1.2 \times DL + 1.6 \times LL \gg 1.2 \times 2262 + 1.6 \times 1464.72 \Rightarrow W_u = 5058$ 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 = $1.2 \text{ hwg} \times \text{bwg} \times \gamma_c \gg 1.2 \times 0.85 \times 0.5 \times 2400 = 1124$ kg/m

Slab portion on Girder/Beam UDL, $sg = w_u$ (slab) \times bwg $\gg 1513.84 \text{ kg/m}^2 \times 0.5 \text{ m} = 756.92$ kg/m

$W_g = \text{UDL, Self weight} + \text{UDL, sg} \gg 1124 \text{ kg/m} + 756.92 \text{ kg/m} \quad W_g = 1980.92$ 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; $S_s = 1.24$ g, $S_1 = 0.5$ g

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; $F_a=1.004$ and $F_v=1.5$

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

$$S_Ms = S_s \times F_a = 1.24 \times 1.004 = 1.245$$

$$S_{M1} = S_1 \times F_v = 0.5 \times 1.5 = 0.75$$

Parameters/factors of design spectral response acceleration;

$$S_{D_s} = 2/3 \times S_{M_s} = 2/3 \times 1.245 = 0.83$$

$$S_{D1} = 2/3 \times S_{M1} = 2/3 \times 0.75 = 0.5$$

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 C_s is equal to;

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

S_{D_s} = is the short period parameter of design spectral response acceleration.

R = response modification factor

I = Factor of importance

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

$$C_s = \frac{S_{D1}}{T(R/I)} \quad \text{For } T \leq T_L$$

$$C_s = \frac{S_{D1} \times T_L}{T^2 (R/I)} \quad \text{For } T > T_L$$

C_s shall not be less than; $C_s = 0.01$

In addition, the building located where S_1 is equal to or greater than 0.6 g, C_s shall not be less than

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

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

So,

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

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

Seismic Base Shear (Section 12.8.1 ASCE);

$$V = C_s \times W$$

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

$$= 25\% \times 488.24 \times 540$$

$$= 65912.4 \text{ Kg}$$

Total, w = 395,312.4 Kg

$$V = C_s \times W$$

$$V = 0.1297 \times 395,312.4 \quad \Rightarrow \quad V = 51272.2 \text{ Kg} = 512.722 \text{ KN}$$

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/m²

Total Design permanent loads; 396+200.2+124.8 » 721 kg/m²

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 $\gamma_f = 1.2$ – at full specified value 2.0 kpa

$$\text{Design live load} = 3 \text{ kpa} = 306 \text{ kg/m}^2 \times 1.2 = 367.2 \text{ kg/m}^2$$

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,

$$\text{Design factored loads} = 1.0 D + 1.0 L \gg 721 + 367.2 = 1088.2 \text{ kg/m}^2$$

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 discontinuous and not restrained. So, ACI moment coefficient analysis method applies.

b) Beam Analysis: For load uniformity the beam size is $h=60$, $b=30$ 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; = $721 \text{ kg/m}^2 \times 3\text{m} = 2163 \text{ kg/m}^2$

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

Design dead load from beam self-weight; = $1.1 \times 432 \text{ kg/m} = 475.2 \text{ kg/m}$

Total design dead load = $2163 + 475.2 = 2638.2 \text{ kg/m}$

Total design live load = $367.2 \text{ kg/m}^2 \times 3 = 1101.6 \text{ kg/m}$

Load combinations:

$$= 1.0 \text{ D.L} + 1.0 \text{ L.L} = 1.0 \times 2638.2 + 1.0 \times 1101.6 = 3739.8 \text{ kg/m}$$

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 $h=100$ cm, $b=50$ cm and loads are summarized as below;

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

$$P = 2 \times W_u \times l_n / 2 \gg 2 \times 3739.8 \times 9.75 / 2 \Rightarrow \mathbf{P = 36463.05 \text{ Kg}}$$

(ii) Factored self-weight of the girder

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

$$\gg 1513.84 \text{ kg/m}^2 \times 0.5\text{m} = 756.92 \text{ kg/m}$$

(iii) Part of slab on girder

$$(U.D.L) S_g = W_u (\text{on slab}) \times b_{wg}$$

$$\gg 1088.2 \text{ kg/m}^2 \times 0.5\text{m} = 544.1 \text{ kg/m}$$

$$W_g = (U.D.L) \text{ self weight} + (U.D.L) S_g \gg 1122 \text{ kg/m} + 544.1 \text{ kg/m}$$

$$\mathbf{W_g = 1666.1 \text{ kg/m}}$$

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;

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

$$C = 9/n+5,$$

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

$$n = 1 \gg C = 9/1+5 = 9/6 = 1.5 \quad \gg \quad \text{So, } C = 1.33$$

$$W = (\text{dead weight of building}) + (25\% \text{ of live load})$$

$$\text{Dead load} = 610 \text{ kg/m}^2 \times (27 \times 20) \gg 610 \times 540 = 329,400 \text{ Kg}$$

$$\text{Dead load} = 329,400 \text{ Kg}$$

$$25\% \text{ of live load} = 25\% \times 306 \text{ kg/m}^2 \times 540 \text{ m}^2$$

$$25\% \text{ of live load} = 41,310 \text{ Kg}$$

$$\text{Total } w = 370,710 \text{ Kg}$$

$$V_b = C \times \alpha_h \times w \gg 1.33 \times 0.12 \times 370,710 \gg \mathbf{0.1596} \times 370,710$$

$$V_b = 59,165.316 \text{ Kg, } \mathbf{V_b = 591.653 \text{ KN}}$$

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

$$V_b = \mathbf{0.1596} \times 395,312.4 \gg V_b = 63091.86 \text{ kg, } \mathbf{V_b = 630.92 \text{ KN}}$$

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/m ² SNiP is 305.92 kg/m ²
At outer mid span	11.74	8.035	46%	
At inner mid span	6.895	4.958	39%	
Negative Moment				Load Combination; IBC; 1.2xDL+1.6x LL SNiP; 1.1xDL+1.2xLL
At outer support	10.24	7.366	39%	
At inner support	9.193	6.611	39%	

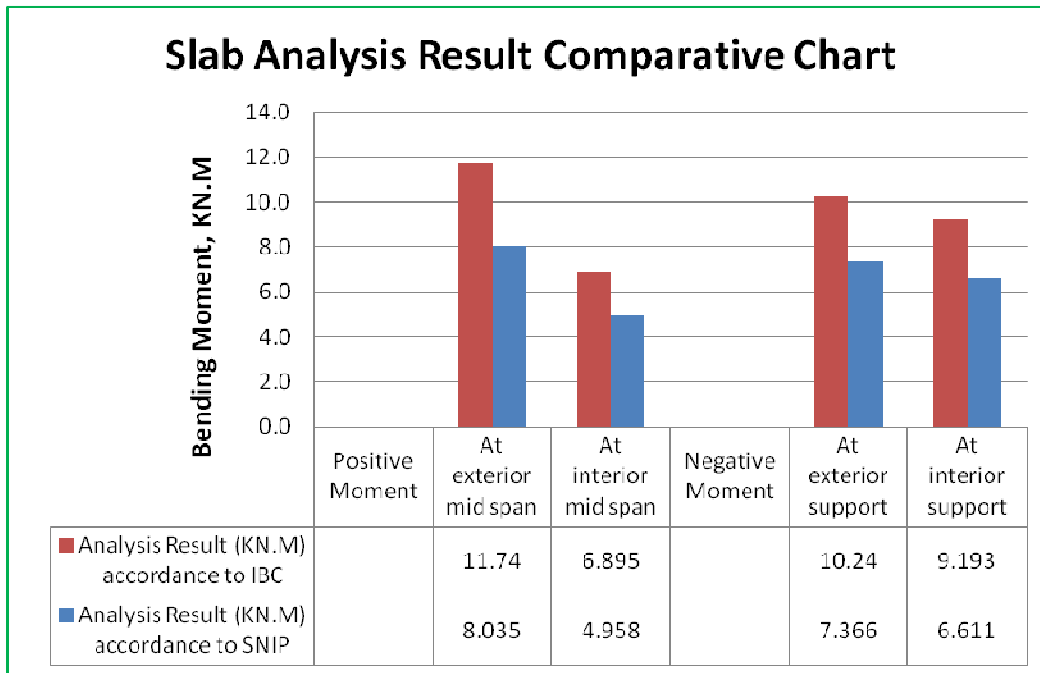


Chart 1. Slab Analysis result comparative illustration accordance IBC and SNiP

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/m ² SNiP; 305.92 kg/m ² Load Combination; IBC; 1.2xDL+1.6x LL SNiP;1.1xDL+1.2xLL
Negative Moment	534.25	395.02	35.25 %	
Positive moment at mid span	437.12	323.2	35.25 %	

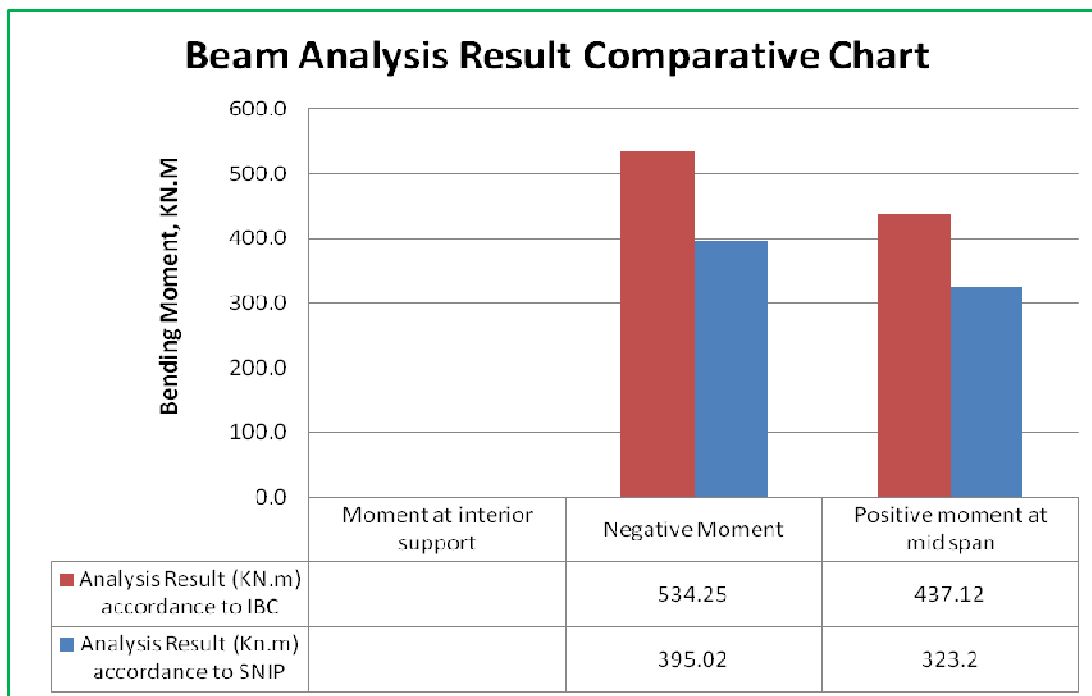


Chart 2. Beam Analysis result comparative illustration accordance IBC and SNiP

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/m ² SNIp; 305.92 kg/m ² 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		

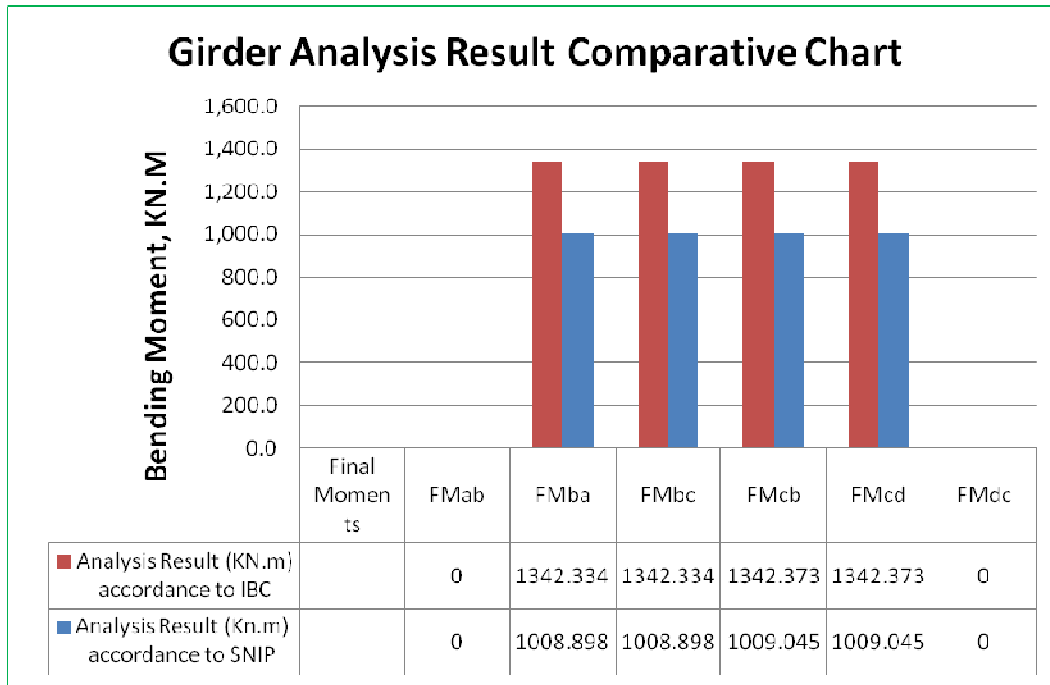


Chart 3. Beam Analysis result comparative illustration accordance IBC and SNIp

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; 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
V	512.722	630.92	-19%	

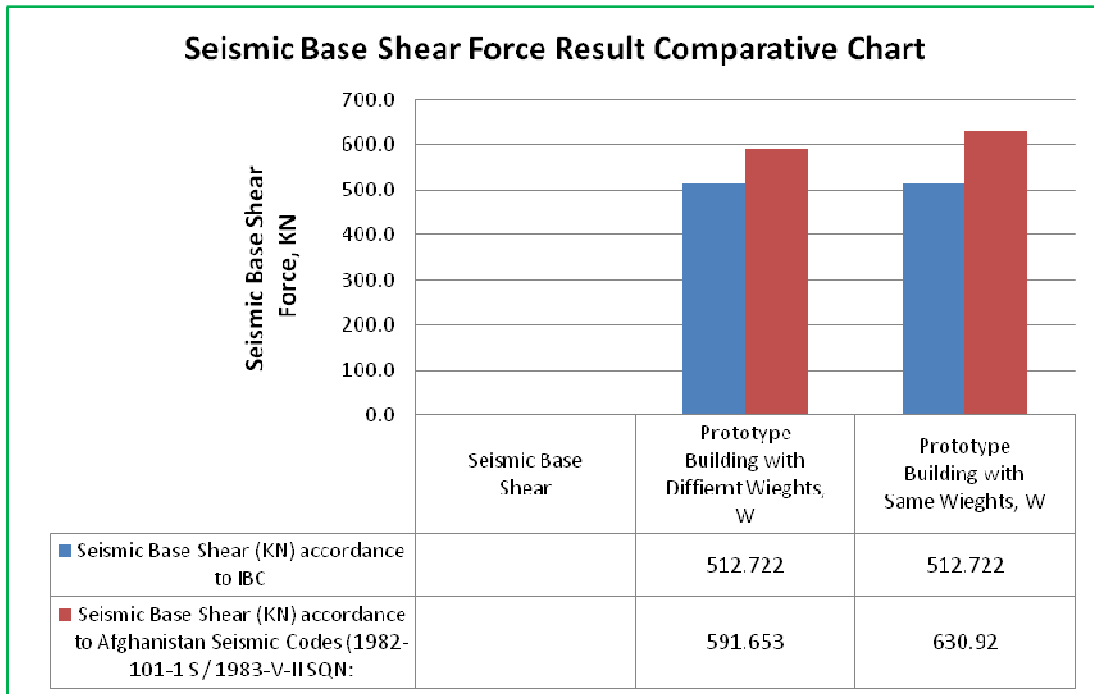


Chart 4. Seismic Base Shear Force Analysis result comparative illustration accordance IBC and Afghanistan Seismic Code based on SNiP

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 f_c'	210	Kgf/cm ²
Reinforcement strength f_y	2,800	Kgf/cm ²

Loads;

Live load (L.L) 100 psf	488.24	kg/m ²	For restaurant hall as per 1607.3 IBC 2009 table 1607.1
Superimposed Dead load (S.D.L)	250.00	kg/m ²	Mud thickness= 8cm and tiles thickness = 5 cm

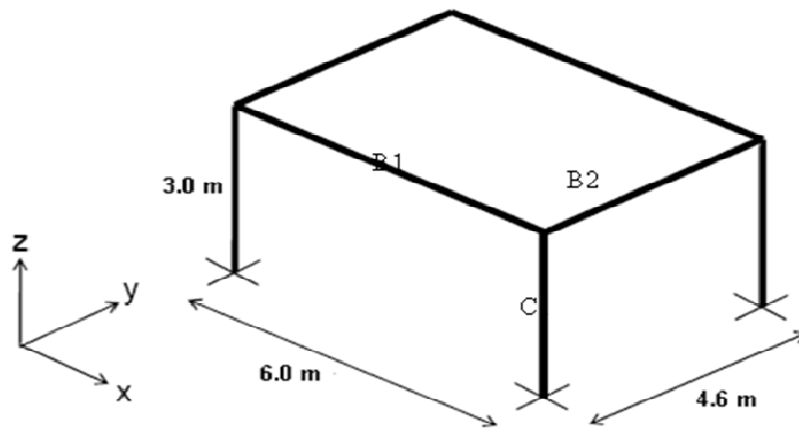


Figure 2. Second prototype building frame arrangements

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;

S _s	1.24	g
S ₁	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 $= (0.15 \times 2400 + 0.08 \times 1922 + 0.05 \times 1922)$

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/m ²)	Self weight of beam(kg/m)	Tributary Area (Ab), m ²	LB (m)	Service dead load per meter (kg/m), DL*Ab/LB	Service Live Load on slab (kg/m ²)	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

$$C_s = S D_s / (R/I)$$

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

$$C_s = S D_1 / T(R/I) \quad \text{for } T \leq T_L$$

$$C_s = S D_1 * T_L / T^2(R/I) \quad \text{for } T > T_L$$

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;

$$C_s = 0.5 * S_1 / (R/I)$$

So, Cs = SDs / (R/I)

$$\text{for } S D_s = 0.83$$

$$R = 8$$

I=II 1 As per table 11.5-1 ASCE, Restaurant Hall with day-care facilities with capacity of more than 150

$$C_s = 0.1037 = C_{98} / (C_{99} / C_{100})$$

Seismic Base Shear (Section 12.8.1 ASCE);

$$V = C_s * W$$

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

$$W_{\text{slab}} + W_{\text{mud}} + W_{\text{tiles}} + W_{\text{beams}} + W_{\text{columns}}$$

$$28,582.54 \quad \text{Kg}$$

25% live load

$$3,368.86 \quad \text{Kg}$$

$$\text{Total } W = 31,951.39 \quad \text{Kg}$$

$$V = C_s * W$$

$$V = 3,314.85 \quad \text{Kg} \quad 48.76 \quad \text{KN}$$

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 $= 4972.28 \text{ kg}/2 =$

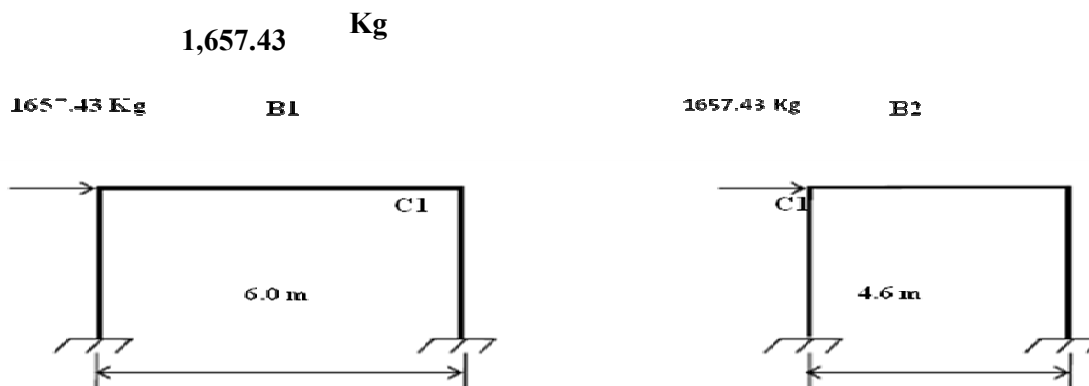


Figure 3. Shear Force application on beams

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	-2339.5	673.7	2143.3	674.8
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

$$\text{Assume } a=hf \quad 15 \text{ cm} \quad \Phi \quad 0.9$$

$$A_s = \frac{M_u}{\{\Phi f_y (d-a/2)\}} \quad 8.17 \text{ cm}^2$$

Recalculate a

$$a = \frac{A_s f_y}{(0.85 f_c' b_{eff})} \quad 0.90 \text{ cm} \quad < hf$$

So, design beam as rectangular beam

Trail #2

$$A_s = \frac{M_u}{\{\Phi f_y (d-a/2)\}} \quad 7.09 \text{ cm}^2$$

$$a = \frac{A_s f_y}{(0.85 f_c' b_{eff})} \quad 0.78 \text{ cm}$$

Trail #3

$$A_s = \frac{M_u}{\{\Phi f_y (d-a/2)\}} \quad 7.08 \text{ cm}^2$$

$$a = \frac{A_s f_y}{(0.85 f_c' b_{eff})} \quad 0.78 \text{ cm}$$

Step (c) **As 7.08 Cm2**
 check for maximum and minimum reinforcement
 $As_{max} = \rho_{max} * bw * d$
 $\rho_{max} = 0.85 * \beta_1 (f_c' / f_y) * (\epsilon_u / (\epsilon_u + \epsilon_y))$
 $\rho_{max} = 0.020$
 $As_{max} = \rho_{max} * bw * d = 32.92 \text{ cm}^2$
 $As_{min} = \rho_{min} * bw * d = 8.10 \text{ cm}^2$
So As = Asmin = 8.10 cm2

β_1 0.85
 ϵ_u
 ϵ_y

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

$M_{ny} / M_{nx} = 0.72 < b/h = 1$

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

$M_{nxo} = M_{nx} + M_{ny} (h/b) * (1 - \beta) / \beta$

where,

M_{nx} and M_{ny} = Biaxial moment strengths

M_{nxo} = Uniaxial moment strengths

β	0.65				
M_{nxo}	3192	Kg.m			
b	30	cm	h	30	cm
f'c	210	kg/cm2			

$P_u = \Phi \{ 0.85 f_c' a b + A_s E * 0.003 (c - d') / c - A_s E * 0.003 * (d - c / c) \}$

Trail #1

assume $A_{st} = 1\%$ of A_g 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

$P_u = \Phi * (0.85 * f_c' * \beta * c * b + A_s * \text{MIN}(f_y; E * 0.003 * (c - d') / c)) - A_s * \text{MIN}(f_y; E * 0.003 * (d - c) / c)$

$$A_s = \frac{\mu - \Phi * 0.425 * f_c' * a * b * (h - a)}{\Phi * \left[\left(\frac{h}{2} - d' \right) (f_{s1} + f_{s2}) \right]}$$

$$f_{s1} = 0.003 * E * (c - d') / c \quad 976.6 \quad \text{kg/cm}^2$$

$$f_{s2} = 0.003 * E * (d - c) / c \quad 2,800.0 \quad \text{kg/cm}^2$$

$$\text{With } a = \beta_1 * c \quad 6.069 \quad \text{cm} \quad \beta_1 = \quad 0.85$$

$$A_s \quad 7.01 \quad \text{cm}^2$$

After four trials the final A_s is found as below

$$A_s \quad 5.64 \quad \text{cm}^2$$

$$\underline{A_{st}} \quad \underline{11.27} \quad \underline{\text{cm}^2}$$

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

Material Properties;

Concrete compressive strength f_c'	210	kgf/cm ²
Reinforcement strength f_y	2,800	kgf/cm ²

Loads;

Live load (L.L) 100 psf	488.24	kg/m ²	<p>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.</p>
Superimposed Dead load (S.D.L)	250.00	kg/m ²	

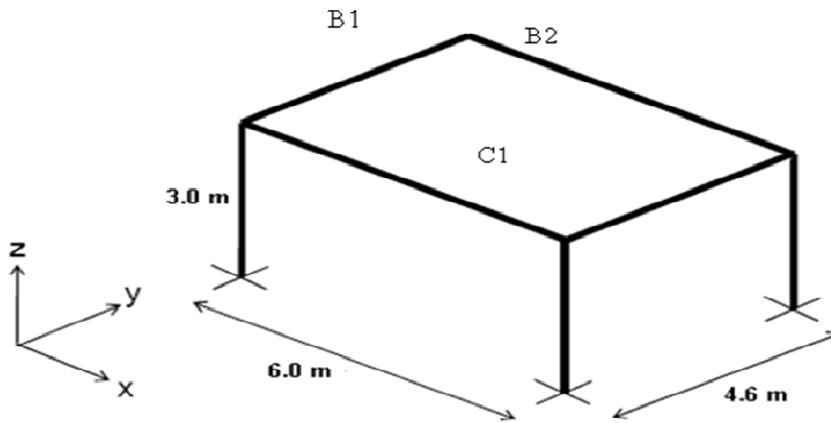


Figure 4. Second prototype building frame arrangements

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.

α_h 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 $= (0.15 \cdot 2400 + 0.08 \cdot 1922 + 0.05 \cdot 1922)$

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
 $= (0.15 \cdot 2400 \cdot 1.1 + 0.08 \cdot 1922 \cdot 1.3 + 0.05 \cdot 1922 \cdot 1.3)$, 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/m ²)	Self weight of beam (kg/m)	Tributary Area (Ab), m ²	LB (m)	Design Service dead load per meter (kg/m), DL*Ab/LB	Design Service Live Load on slab (kg/m ²)	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 $\alpha_h=0.12$ with respect to soil class of the site

α_h 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;

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

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

α_h 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

V_b	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 = $5751.25 \text{ kg}/2 = 2,875.63 \text{ Kg}$

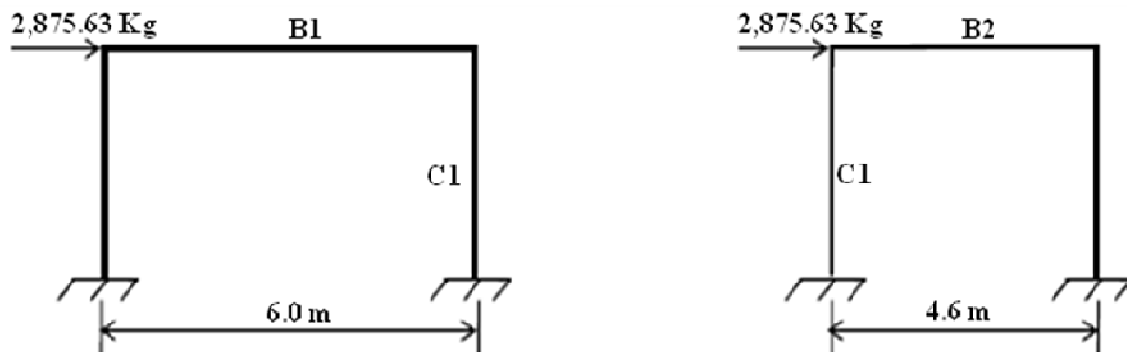


Figure 5. Shear Force application on beams

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 $hf \geq 0.1h=6cm$, 1/2 of clear distance b/w longitudinal ribs 270 cm

So b_f 200 cm

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

Steel reinforcement Class A-II 280 Mpa (equivalent to 40,000 psi) , for this the $R_s = 280$ 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

$R_b \cdot b' \cdot f' \cdot h' \cdot (h_0 - 0.5h'f)$ 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) = $M/R_b \cdot b \cdot h_0^2$

$\alpha_m = M/R_b \cdot b \cdot h_0^2$ 0.0179 < $\alpha_R=0.449$ 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 $\alpha_m=0.0182$ we find the value of $\zeta=0.990$

$A_s=M / (R_s \cdot \zeta \cdot h_0)$ 5.390 cm2

Check for Minimum steel requirement

$A_{smin}=\mu \cdot b \cdot h_0/100$ $\mu=0.1\%$ 0.1 For $\zeta=0.990$ According Annex 2

A_{smin} 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

M_{ny} / M_{nx} 0.78 < b/h 1

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

$M_{nxo} = M_{nx} + M_{ny} (h/b) \cdot (1-\beta)/\beta$

where,

M_{nx} and M_{ny} = Biaxial moment strengths

M_{nxo} = Uniaxial moment strengths

β 0.65

M_{nxo} 4613 Kg.m

Effective Length of the column

$$l_0 = 1.2 * 3 \text{ m} = 3.6 \text{ m}$$

$$3.6 \text{ m}$$

[SNiP 2.03.01-84 Section 3.25] and [Guide to SNiP 2.03.01-84 Table 13]

Column Section

$$\text{Width (b)} \quad 30 \text{ cm}$$

$$\text{Height (h)} \quad 30 \text{ cm}$$

Area of compressed or tensile reinforcement (for symmetric arrangement)

$$A_s = \text{if}[\alpha_n < \xi_R, R_b * b * h_0 / R_s * \alpha_m 1 - \alpha_n * (1 - \alpha_n / 2) / 1 - \delta, R_b * b * h_0 / R_s * \alpha_m 1 - \xi * (1 - \xi / 2) / 1 - \delta]$$

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

$$\text{as } \alpha_n < \xi_R, \text{ So } A_s = (R_b * b * h_0 / R_s) * (\alpha_m 1 - (\alpha_n * (1 - \alpha_n / 2))) / 1 - \delta$$

$$6.7084 \text{ cm}^2$$

Actual reinforcement ratio

$$\mu_{\text{actual}} = A_s * 2 / b * h_0 \quad 0.018$$

$$\text{Gyration radius } i \quad 8.66$$

$$\text{Slenderness ratio } \lambda \quad 41.57$$

Minimal Reinforcing ratio

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

$$\mu_{\text{minimul}} = \text{if}(\lambda < 17, 0.1, \text{if}(\lambda \geq 17 \wedge \lambda \leq 35, 0.2, \text{if}(\lambda > 35 \wedge \lambda \leq 83, 0.4, 0.5))) / 100$$

$$\mu_{\text{minimul}} = \quad 0.004$$

[Guide to SNiP 2.03.01-84 Table 47]

Total Reinforcement Area shall not be less than the minimum value

$$A_{\text{total}} = \text{if}(\mu_{\text{actual}} < \mu_{\text{minimul}}, \mu_{\text{minimul}} * b * h_0, 2 * A_s)$$

$$\mathbf{A_{total} \quad 13.42 \quad cm^2}$$

Design Result Comparison:

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

Prototype Building	Design of Building by IBC 2009, ACI 318M-08	Design of Building according SNIP 2.0.07-89 / SNIP II-7-81	How much bigger/smaller IBC than SNiP (IBC-SNiP/SNiP) %	Main reasons caused the differences
Steel Reinforcement in Cm2 for Original Building Analysis Moments				Building Weight;
				IBC W; 395,312.4 Kg, SNiP; 395,312.4 Kg
				Importance Factor;
				IBC; 1, SNiP;1
				Seismic Coefficient;
				IBC=0.104 SNiP=0.180
				Load Combination:
				IBC = 1.2D+1.6L SNiP = 1.0 D+1.0 L
				Beam Moment, Kg.m
				IBC = 9570.90 SNiP = 8212.80
				Column Uniaxial Moment, Kg.m
				IBC = 3191.85 SNiP = 4612.71
				Column Axial Load, Kg
				IBC = 10439.85 SNiP = 7576.89
a, cm				
a= As*fy/ (0.85 fc'*beff) = 0.78cm				
am = M/Rb*b*ho2				
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
				Column Axial Load, Kg
Beam	14.92	13.33	11.97%	
Column	24.51	14.48	69.24%	IBC = 15000 SNiP = 15000

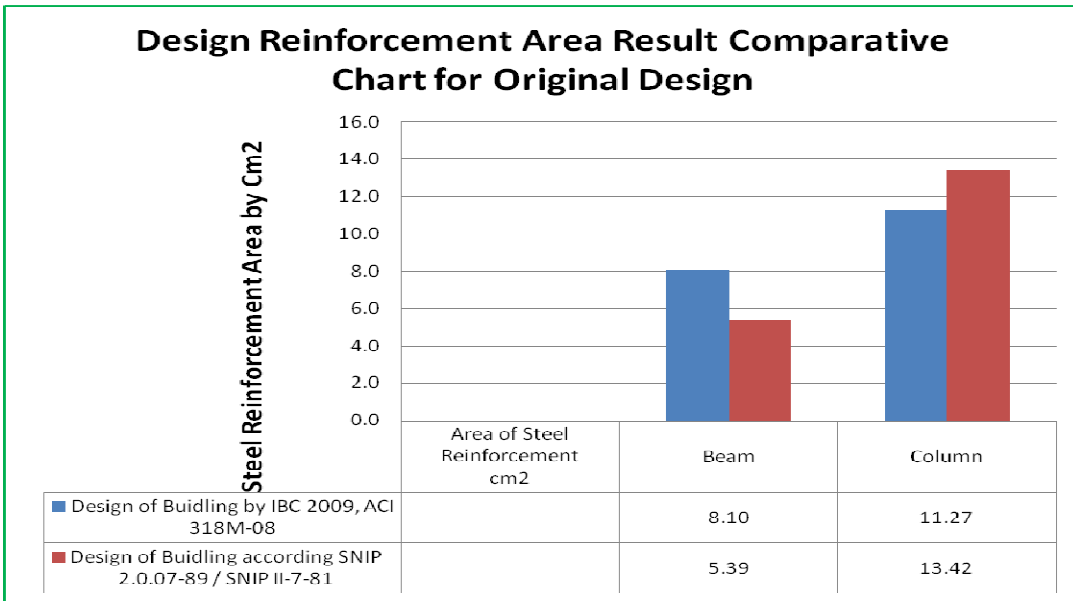


Chart 5. Design result comparative illustration according IBC and SNiP

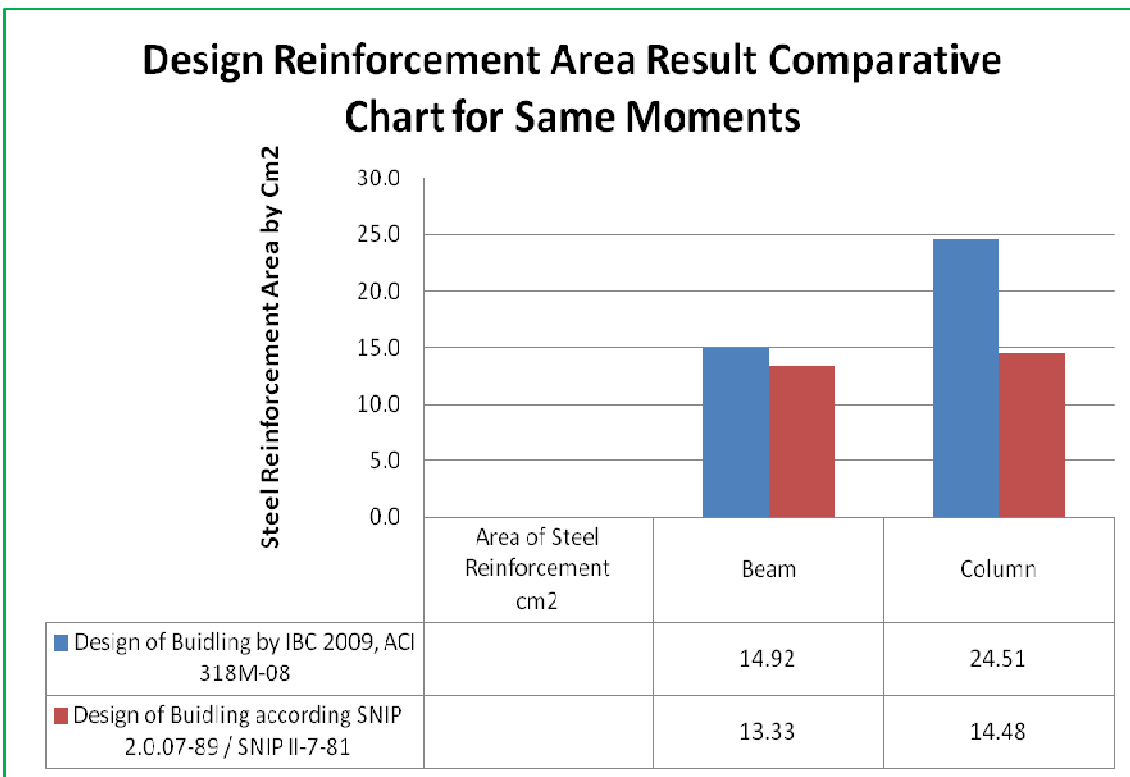


Chart 6. Design result comparative illustration according IBC and SNiP

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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