National Sports Council

Dasarath Rangasala

Tripureswor, Kathmandu

Research Report on

Seismic Vulnerability Assessment of Dasarath Rangasala

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Center of Resilient Development (CoRD)

Harihar Bhavan (Opposite UN Gate),

Lalitpur, Nepal

PO Box: 1681

Phone No.: 977-01-5010268, 977-01-5010032

Website: www.cordvia.org

Email: info@cordvia.org

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1. GENERAL AND BACKGROUND

1.1 BACKGROUND

Nepal is among the most disaster prone countries in the world. The country is ranked 11th in earthquake vulnerability, and Kathmandu is said to be exposed to the greatest earthquake risk among 21 megacities around the world. It had devastating earthquakes in 1934 and 1988.The earthquake of 1934 measuring 8.4 on the Richter scale is estimated to have killed over 16,000 people in Nepal and India. It caused extensive damage in Nepal: Over 8,500 lives were lost; over 80,000 houses were completely damaged, and over 126,000 houses were severely damaged, majority of them were of Kathmandu valley. The more recent earthquake of magnitude 6.6 in Udayapur district in 1988 killed 721 people and destroyed 64,476 houses.

Dasarath Stadium, a multi-purpose stadium is located in Tripureswor, Kathmandu, Nepal. It is the biggest stadium in Nepal. The stadium has a capacity to hold 25000 spectators including 5000 fixed seats in the main stand in the west side of the stadium. Built in 1961 AD, most of the national and international sport events and cultural and entertainment programs are held in this stadium.

Being located in Kathmandu, one of the highly seismic prone cities among 27 megacities of the world makes the stadium very susceptible to earthquakes. Furthermore, the structures of stadium are built more than 5 decades ago, which is the normal life of any concrete structures. As Kathmandu is potential to earthquake, stadium may put the people lives at risk. So it is high time to assess the seismic vulnerability of the facilities to ensure the smooth functioning of one of the iconic landmark of Nepal for national and international events.

Hence, the existing infrastructure should be subjected to vulnerability assessment tools in order to identify their strength, resilience and their vulnerability when subjected to a major earthquake.

1.2 OBJECTIVE

Stadiums and sports arenas are particularly sensitive areas prone to mass casualties and stampede when packed to full capacity. Therefore, the proposed work is carried out to identify the potential seismic hazards to the existing structures and to brief about the present status, and if required recommend for the need of detail structural assessment of the structures.

Specific objectives are:

- Evaluate the structural status of the existing structures by using qualitative method of the vulnerability assessment of structures
- Check for the functional requirement of the stadium structure during crunch hours in context of Disaster Risk Reduction (DRR).

1.3 SCOPE OF WORK

The scope of the work for assessment is as follows:

I. Conduct a survey to determine the structural characteristics of the stadium.

II. Review the existing documents mainly the drawings

III. Assess the structural earthquake vulnerability of the buildings by qualitative method.

IV. Carrying out the Non-destructive tests of the existing structures and performing a quality check on their current status.

V. Determine the structural status of structures and recommend whether detail structural vulnerability analysis is required or not.

1.4 CONCEPT OF RESILIENT STADIUM

A disaster resilient stadium or sports arenas is the one which not only is safe during disasters but also operational and serves as shelter to the communities in the aftermath of disasters. There are four major factors that influence directly or indirectly the continuous operation of the stadiums whenever an earthquake occurs. Although these factors are relevant equally for most of the disasters, further discussion is carried out with focus on seismic disaster. The four major factors are:

i. Location damage

ii. Structural damage

iii. Non-structural damage

iv. Functional collapse

The condition of non-functioning of a stadium and interaction of the above four issues are shown in Figure 1.

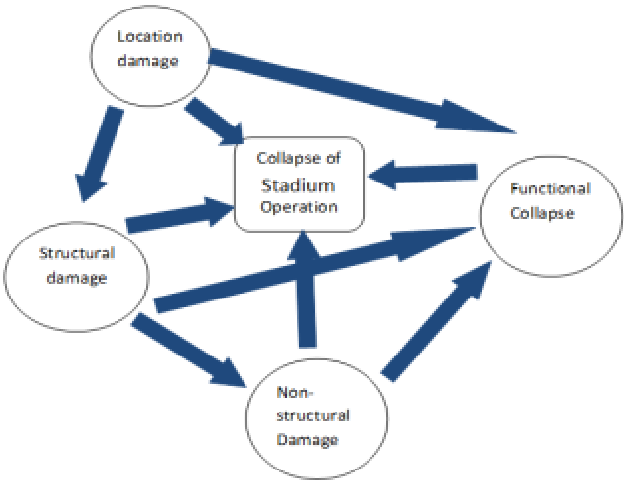


Figure 1-1 Reasons for non-functioning of Stadiums

Non-compliance of performance level in any one or combination of the above issues may result in one or more of the following scenarios:

i. Stadium itself suffering severe damage and loss

ii. Injury or loss of lives of the occupants

Location safety implies that Stadiums are constructed in places which are not affected adversely by disasters. Construction of Stadium in landslide susceptible areas, in flood plains, in liquefaction potential area and areas where fire hazard is significant may result in severe adverse consequences to Stadium during disasters. Structural safety is one of the major concerns of resilient Stadiums during earthquakes. A properly planned, well designed and constructed stadium can sustain major earthquakes without significant damage to its components. Structural damage may result in closure of stadiums in emergencies and may also lead to injury and loss of lives of huge numbers of spectators and/or stadium personnel (staffs).

1.5 OVERALL METHODOLOGY

The study consists of qualitative approach of structural evaluation method. The qualitative evaluation estimates structural vulnerability based on visual inspection, and review of drawings and checking of codal provisions. Also the material strength is estimated upon by the Non-destructive tests being conducted at the site after identifying various vulnerable locations. The general procedures for seismic vulnerability estimation of existing buildings proposed are site visit and data collection; selection and review of evaluation statements; follow-up fieldwork; and analysis of buildings by qualitative approach.

The overall methodology adopted for this study is as follows:

i. Reconnaissance survey of the Stadium structure

ii. Identification of the Stadium typology based on construction materials and structural systems

iii. Checking codal requirements with the current status of the stadium using simple analytical calculations, so far possible.

iv. Detailed visual survey of the Stadium which includes:

- Identification of damages and cracks
- Identification of structural vulnerability factors: Plan and vertical irregularities, vertical load path, configuration problems, lateral force resisting system, material deterioration etc.

v. Identification of the design criteria and structural system, and carrying out the non-destructive tests of the vulnerable locations so as to get strength of material of existing structures.

vi. Carrying out a brief functional check for the better performance of the stadium during rush hours and eliminate any chance of havoc during audience-packed hours.

vii. Summarization of findings - status of structure and recommendation

The detail description of each methodology for seismic vulnerability assessment is given in respective chapters.

2. PRELIMINARY SURVEY

During the preliminary survey consultation meeting was conducted with the concerned personnel of the Stadium about the procedures of the assessment and available drawings and documents related to the stadium were collected. Information of the stadium such as date of construction, its age, use, soil type and geological condition, structural system, architectural and structural characteristic, designed capacity and other relevant data were noted during the pre visit.

2.1 LOCATION OF BUILDINGS IN SEISMIC HAZARD MAP

As per IS 1893:2002 (Part 1), Nepal lies in high seismic risk (Zones IV and V) as shown in Figure 2-1. The details of different seismic zones are given the following Table 2-1:

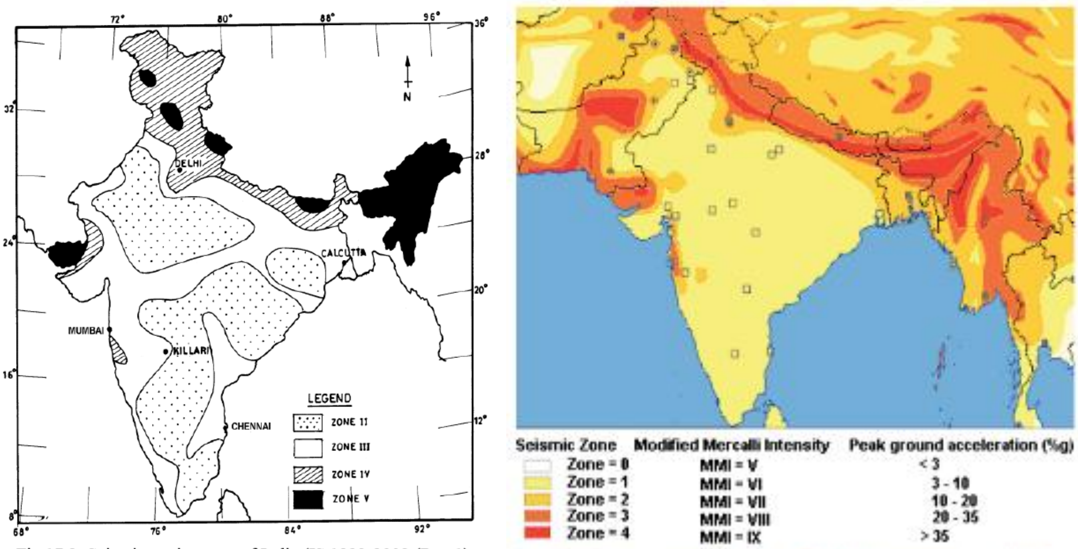


Fig 17.2: Seismic zoning map of India (IS 1893-2002 (Part 1).

Source: Global Seismic Hazard Assessment Program (GSHAP), http://www.seismo.ethz.ch/GSHAP/

Figure 2-1 Seismic zones

Table 2-1 Seismic Zone of India

Zone II	Low seismic hazard (maximum damage during earthquake may be up to MSK intensity VI)
Zone III	Moderate seismic hazard (maximum damage during earthquake may be up to MSK intensity VII)
Zone IV	High seismic hazard (maximum damage during earthquake may be up to MSK intensity VIII)

Zone V	Very high seismic hazard (maximum damage during earthquake may be of MSK intensity IX or greater)

As per Nepal Building Code, the seismic zoning of Nepal is as shown in the following Figure 2-2. Z is the seismic zoning factor that divides the country into fives zones for the purpose of seismic design of buildings with the values ranging from 0.8 to 1.1. The assessed stadium is located in the seismic zoning factor, Z of 1.0.

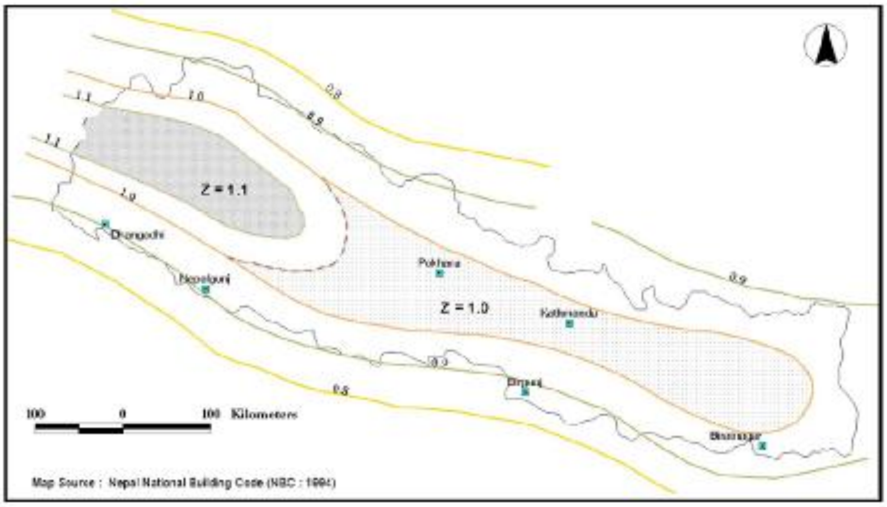


Figure 2-2 Value of seismic zone, Z (NBC 105)

2.2 GENERAL INFORMATION ABOUT STADIUM

Table 2-2 Information Summary

Name of the stadium	Dasarath Stadium
Year of Construction	1961 A.D.
Name of the owner	National Sports Council
Purpose of the stadium	Organize multi-disciplinary events (sports and cultural)
Total No. of blocks	10 (including VIP Block)
Estimated Capacity	25,000 max.
Area of the Stadium	22,400 m^2 (Approx)

2.3 REVIEW OF DOCUMENTS AND DRAWINGS

As-built architectural drawings that were developed in 2056 BS by Krishna, Parera, and C.E. JV were collected, reviewed and verified in the site. Alterations and deviations were not observed during the visit.

3. CODAL COMPATIBILITY

The codes define a certain set of guidelines which every building or structure has to follow in order to ensure the safety of the structure. The negligence or incompliance with the codal provisions may result in catastrophic results. The codal compatibility regarding the present strength and other determining criterion that are commonly used in the design portion these days was designed and built were investigated in this section.

3.1 REVISION OF THE CODAL PROVISIONS

Although the exact code which was used to design the stadium is unknown, it would not be wrong to say that the codes have come a long way since then. Several provisions in the codes have been modified in Indian Standard, British Standard or European or American since the construction of the stadium.

Indian Code:

Five revisions have been made in the Indian Code since its development around 1962. The latest code of practice for the construction design of structures to mitigate seismic vulnerability has made few changes when compared with their previous versions.

Some of the significant modifications made in the codes are as follows:

- The seismic zone map now contains only four zones as compared to the five zones earlier, and the relative values of zone factors are now different.
- The code now provides realistic values of acceleration from which the design forces are obtained by dividing the elastic forces by a response reduction factor; this enables a clear statement of intent to the designer that the design seismic force is much lower than what can be expected in the event of a strong shaking.
- The design spectrum shape now depends on the type of soil and the foundation-soil factor (b) has been dropped.
- The code now requires that there be a minimum design force based on empirical fundamental period of the building even if the dynamic analysis gives a very high value of natural period and thus low seismic force.

European Code

The European code is scheduled to revision process every 5-10 years under normal circumstances.

In addition to it, non-scheduled data revisions are, by definition, not announced in advance either because they are a result of unforeseeable events such as errors or accidents.

American Code

The **Uniform Building Code** was first enacted by the Pacific Coast Building Officials (now International Conference of Building Officials (ICBO)) on October 18-21, 1927. Since then the revised editions of this code are published approximately every 3 years.

After the stadium was built, the **California Building Standards Code** (California Code of Regulations, Title 24) was created in 1978 as an amalgamation and reorganization of existing codes. Then in 2000 the new national code called international building code was published by international Code council.

3.2 AVAILABILITY OF DESIGN DATA

The design data, methodology of the design of the stadium are not available for reference and it is not known which code or design procedure was used for the design calculations. So on the basis of the Non-Destructive tests, a reasonable assumption can be developed regarding the design data.

4. PHYSICAL DAMAGE ASSESSMENT

4.1 PHYSICAL INSPECTION

The exterior and interior of the different blocks and structures were inspected and collected the primary and secondary information by physical inspection, interview and material exploration regarding technical details of the structures.

The nature and intensity of the damage/defects were thoroughly examined for each of the structures/blocks. Different cracks (horizontal, vertical, diagonal, stepped), were observed in the site and were recorded.

4.2 CHECK FOR SLENDERNESS RATIO

The structure seems to be composite with around half of the portion of the blocks completely Reinforced Concrete (RC) Structure and half of the block as load bearing masonry structure. There by slenderness ration is checked here to examine the safety of the infill and load bearing walls against the out-of-plane failure.

For a wall, slenderness ratio shall be effective height divided by effective thickness or effective length divided by the effective thickness, whichever is less.

Table 4-1 Slenderness for different walls of blocks

Blocks	Walls	**Max. Slenderness Ratio for walls**		
		Existing	Recommended	Remarks
A	Along Radial Direction	27	27	SAFE
	Along Circumferential Direction	27	27	SAFE
B, C, D, E	Along Radial Direction	27	27	SAFE
	Along Circumferential Direction	27	27	SAFE
F	Along Radial Direction	22	27	SAFE

	Along Circumferential Direction	60	27	UNSAFE
G	Along Radial Direction	21	27	SAFE
	Along Circumferential Direction	27	27	SAFE
H	Along Radial Direction	27	27	SAFE
	Along Circumferential Direction	27	27	SAFE
I	Along Radial Direction	27	27	SAFE
	Along Circumferential Direction	27	27	SAFE

4.3 STRENGTH CRITERION

Strength of concrete along with its durability plays an important role in the vulnerability of the building and also in determining the present condition of the concrete. Some of the important features of the concrete were investigated thoroughly during the assessment.

4.3.1 Strength Variation with Age

According to the Static, dynamic and low cycle fatigue testing of 20 - 30 years old concrete carried out around the world, dynamic strengthening factors were 2 - 4 times lower than those of 28 day concrete. Dynamic strengthening in splitting has been approximately 4 times smaller than that in compression. Modulus of elasticity is increasing with age more significantly than the strength. Elasticity modulus increase has been observed even in cases when there was no increase of strength. Ultimate strains were drastically lower than those of a young concrete. A reduction of approximately 50% has been observed. Up to the stress of 75% of a peak value, old concrete behaves as an elastic material.

All these changes in properties are on the alarmingly unsafe side in terms of seismic performance. Concrete with age is evidently getting to be more rigid, less ductile and exhibits a very unhappy tendency to brittle explosive modes of failure. It is becoming seismically fragile and is an easy target for seismic forces. Old concrete itself is an object of destruction and it triggers damage and destruction of other elements. Contribution of old concrete in resisting shear and torsion is diminishing drastically. Due to a very limited ability to expand laterally, the effectiveness of confinement is also reducing and ultimately ceases to have any positive effect. Low cycle fatigue capacity, i.e. seismic capacity, is totally dependent on the available reserve of plastic deformations. Only concrete with a remaining capacity to undergo plastic deformation is able to develop dynamic strengthening and provide a predictable resistance to cyclic loading. Large strength increases, commonly adopted in seismic analysis are absolutely not relevant to the actual phenomenon.

Right from this an extremely important conclusion can be drawn. Somewhere between 25 and 30 years of service life, concrete can develop its ultimate value of residual strains, can become brittle and lose its ability for the dynamic strengthening and drastically reduce its fatigue resistance.

4.3.2 Strength variation due to Creep and fatigue

The fatigue strength of a material is defined as the maximum stress which the material can sustain for a given number of cycles. It decreases with number of cycles and is considerably lower than their static strength. The fatigue limit or endurance limit corresponds to the maximum stress which the material can sustain for an infinite number of cycles.

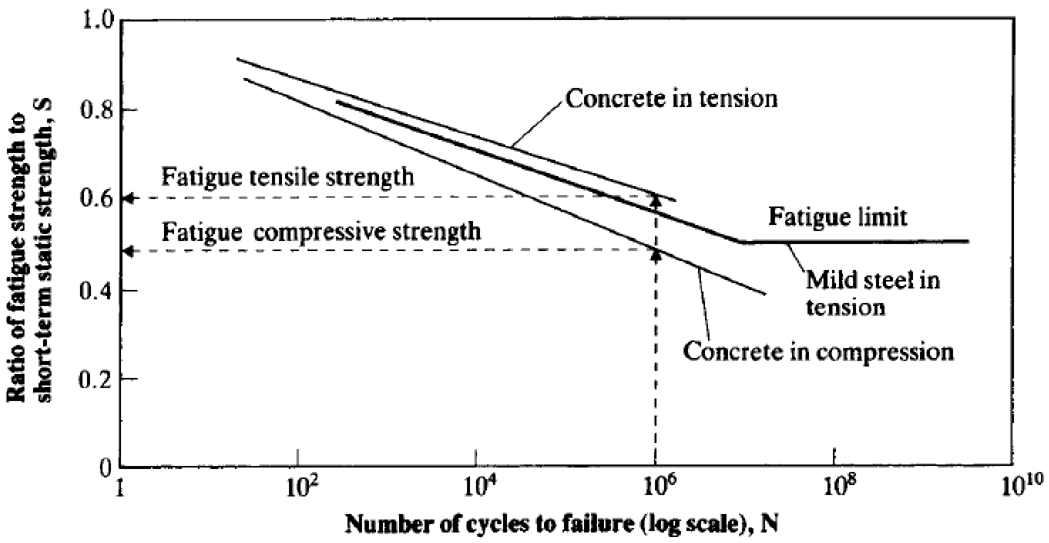


Figure 4-1 Typical relation between fatigue strength and No. of Cycles

Fatigue occurs when a material is subjected to repetitive loading and unloading. Eventually a crack will reach a critical size, and the structure will suddenly fracture. The shape of the structure will significantly affect the fatigue life; square holes or sharp corners will lead to elevated local stresses where fatigue cracks can initiate. Round holes and smooth transitions or fillets are therefore important to increase the fatigue strength of the structure.

In the Modal Code (MC) 2010, analytical expressions are given to estimate the number of cycles to failure, N, for a constant minimum and maximum stress level for pure compression, compression-tension and pure tension, respectively. In these relations the maximum and minimum stress levels for compression $S_{c,max}$ and $S_{c,min}$ are defined as given in eqs. (I and II).

In this equation the coefficient $\beta_{c,sus}(t, to)$ takes into account the effect of high sustained loads in cases where the mean stress during fatigue loading is high. A relation for $\beta_{c,sus}(t, to)$ is given in MC2010. The product $\beta_{cc}(t).\beta_{c,sus}(t, to)$ may also be taken from Figure 4-2 i.e. $f_{cm,sus}(t,t_0)/f_{cm} = \beta_{cc}(t).\ \beta_{c,sus}(t,t_0)$. For an age at loading of 28 days $f_{ck,fat}$ decreases from about 0.82fck for a low strength grade to about 0.75fck for a high strength grade.

$$S_{c,\,max} = |\sigma_{c,\,max}| / f_{ck,\,fat} \qquad \textbf{(I)}$$

$$S_{c,\,min} = |\sigma_{c,\,min}| / f_{ck,\,fat} \qquad \textbf{(II)}$$

With

$$f_{ck,\,fat} = \beta_{cc}(t)\, \beta_{c,sus}(t, to)\, f_{ck}\, [\,1 - f_{ck} / (25 f_{cko}\;)] \qquad \textbf{(III)}$$

Where:

$S_{c,max}$	maximum stress level
$S_{c,min}$	minimum stress level
$\sigma_{c,max}$	maximum compressive stress [MPa]
$\sigma_{c,min}$	minimum compressive stress [MPa]
f_{ck}	characteristic compressive strength [MPa]
$f_{ck,fat}$	fatigue reference compressive strength [MPa]
f_{cko}	= 10 MPa
$\beta_{cc(t)}$	coefficient to take into account the effect of age at the beginning of fatigue loading on the compressive strength of the concrete, see eq. (IV)

$\beta_{c,sus}(t, to)$ coefficient to take into account the effect of high mean stresses during fatigue loading

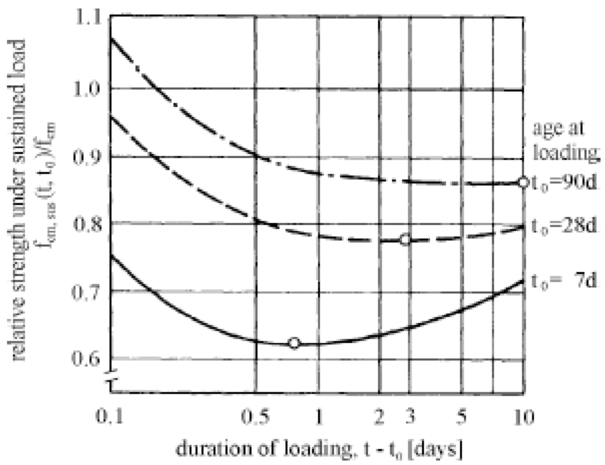


Figure 4-2 Compressive strength of concrete subjected to sustained high overloads according to the relations given in MC2010

The rate at which the concrete strength increases with time depends on a variety of parameters, in particular the type and strength class of the cement, the type and amount of admixtures and additions, the water/cement ratio and environmental conditions. The development of the compressive strength with time may be estimated from equations below.

$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm}$$

$$\beta_{cc}(t) = \exp\left[s.\left\{1-\left(\frac{28}{t/t_1}\right)^{0.5}\right\}\right] \qquad \textbf{(IV)}$$

Where:

$f_{cm}(t)$ mean compressive strength [MPa] at a concrete age t [days]

f_{cm} mean compressive strength [MPa] at a concrete age of 28 days

$\beta_{cc}(t)$ function to describe the development of compressive strength with time

t concrete age [days]

t_1 = 1 day

s coefficient which depends on the strength class of cement as given in Table below.

Table 4-2 Coefficient against Grade of Cement

Strength class of cement (Grade)	43	53
s	0.25	0.2

Sample Calculation:

For M25 Concrete (assumed), we intend to calculate the fatigue strength of the concrete after 20 years. For this purpose we assume the Grade 53 cement being used for the design calculation.

Now,

s = 0.2

t = 20 x 365 days = 20 years.

$\beta_{cc}(t) = 1.2$

$\beta_{cc}(t)\, \beta_{c,sus}(t, to) = f_{cm,sus}(t,t_0)/f_{cm} = 0.8$

$\Rightarrow \beta_{c,sus}(t, to) = 0.67$

$f_{ck, fat}$ = 18 Mpa.

It can be concluded that the strength under normal assumptions is decreased as much as by 28% of the initial static strength and the concrete is just aged 20 years. However, the real age of the concrete is far greater than the assumed value and thus the strength might correspond to the lower values than calculated above.

4.4 DURABILITY OF THE CONCRETE

Durability is a major concern for concrete structures exposed to aggressive environments. Many environmental phenomena are known to significantly influence the durability of reinforced concrete structures. Carbonation is one of the factors to affect the concrete durability.

Carbonation is the reaction of the hydration products dissolved in the pore water with the carbon dioxide in the air which reduces the pH of concrete pore solution from 12.6 to less than 9 and steel passive oxide film may be destroyed and accelerating uniform corrosion.

Carbonation-induced corrosion can increase crack development and decrease concrete durability.

No Portland cement is resistant to attack by acids. In damp conditions, Sulfur dioxide (SO_2) and carbon dioxide (CO_2), as well as some other fumes present in atmosphere, form acids which attack concrete by dissolving and removing a part, of the hydrated cement paste and leave a soft and very weak mass. This form of attack is encountered in various industrial conditions, such as chimneys, and in some agricultural conditions, such as floors of diaries.

During the site visit, seepage problem were observed at various locations especially at joints and walls. This makes the concrete carbonation inevitable. Thus there is a serious question as far as the durability of the concrete is considered. The overall condition of the concrete found is in poor condition and immediate measures are recommended for the rehabilitation of the concrete.

4.5 SUMMARY OF DEFFECTS OBSERVED

Table 4-3 Summary sheet of defects

Block ID	Component	Defects Observed	Location	Possible Reason	Photo Id	Remarks
VIP/ VVIP	Slab	Cracks	Top floor	Improper drainage of water	VIP1	Slab Deterioration
		Concrete Spalling	Top floor	Improper drainage of water	VIP2	Slab Deterioration
		Corrosion of Rebar	Top floor	Improper drainage of water	VIP2	Slab Deterioration
		Seepage	Top floor	Improper drainage of water	VIP1/ VIP2	Slab Deterioration
		Plants seen	Top floor/Bottom Floor	Lack of Maintenance	VIP3	Slab Deterioration
	Columns	Cracks	Ground floor	Lack of Maintenance	VIP4	Column Crushing
		Concrete Spalling	Ground Floor	Lack of Maintenance	VIP4	Column Crushing
		Corrosion	Ground Floor	Lack of	VIP4	Rebar Exposed

		of Rebar		Maintenance		
Block D	Slab	Concrete Spalling	Ground Floor	Improper drainage	D4	Rebar exposed
		Corrosion of Rebar	Ground Floor	Improper drainage	D4	Rebar exposed
		Seepage	Ground Floor	Improper drainage	D3/D4	Non-Structural damage
	Columns	Cracks	First floor	Improper drainage	D2	Non-Structural damage
		Plaster Spalling	First floor	Improper drainage	D2	Non-Structural damage
		Seepage	First floor	Improper drainage	D2	Non-Structural damage
	Walls	Plaster damage	First floor	Improper drainage	D1	Non-Structural damage
		Seepage	First floor	Improper drainage	D1	Non-Structural damage
Block F	Slab	Cracks	First floor	Improper drainage	F4	Damaged concrete
		Concrete Spalling	First floor	Improper drainage	F4	Damaged concrete
		Corrosion of Rebar	First floor	Improper drainage	F4	Rebar exposed
		Seepage	First floor	Improper drainage	F4	Reason for concrete deterioration
Block G	Columns	Cracks	Poor concreting	First floor	G3	Concrete deterioration
	Beams	Wall separation	Poor concreting	First floor	G1	Concrete deterioration
		Crack at interface	Poor concreting	First floor	G3	Beam/column interface
	Walls	Cracks	Overload/differential settlement	Ground floor	G2	Horizontal cracks seen
		Seepage	Improper drainage	First floor	G2	Reason for concrete deterioration


Photo=VIP1: Seepage Problem


Photo= VIP 2: Concrete Spalling, Rebar Corrosion


Photo=VIP3; Trees Seen


Photo= VIP4; Concrete spalling/ cracks seen


Photo=G1: Wall separation under beam


Photo= G2: Cracks seen in the walls.


Photo=G3; Cracks seen at beam-column interface


Photo= F4; Concrete spalling/ Rebars Exposed


Photo=D1: Seepage Problem

Photo= D2: Maintenance and seepage Problem

PhotoD3; Seepage at Joint

Photo= D4; Concrete spalling/ Rebars Exposed

Photo=J1: Joint Problem

Photo= J2: Joint Problem

Photo=J3; Joint Problem

4.6 SUMMARY OF OBSERVATION

The seepage problem is found to be significant at most of the locations especially at the joints. The joints are not properly sealed as a result of which the water from the top seating area is getting drained straight into the walls and slabs. This affects structure adversely and in result affects the quality of the concrete and is causing corrosion on steel bars. The block D where there are restroom facilities for the officials and the athletes is found to bear the same problem.

The seepage has caused spalling of the concrete and carbonation of the existing rebars significantly and is spreading rapidly. The rebar have been exposed at few locations and corrosion has been observed on it. There is an urgent need to improve the drainage facility by sealing the concrete surface against any water seepage.

Most of the walls are free from cracks and found in good condition during physical inspection. The cracks are observed at the corners where two walls meet and also at the slab-wall junction. Damp patches are found in the walls resulting in the spalling of the concrete and plasters to crack. There is an urgent need of repair and maintenance work to correct the defects on walls and joints.

4.7 IDENTIFICATION OF VULNERABILITY FACTORS

The different vulnerability factors associated with particular type of building are checked with a set of appropriate checklist from FEMA 310, "Handbook for the Seismic Evaluation of Buildings" and Indian Standard Guidelines for Seismic Evaluation and Strengthening of Existing Buildings. The basic vulnerability factors related to the building system, plan irregularities, vertical irregularities, lateral force resisting system, connections diaphragms etc. are evaluated based on visual inspection and review of drawings provided. The checklist used for checking different vulnerability factors of the assessed building is given in ANNEX I.

The influences of different vulnerability factors to the buildings on the basis of visual inspection for the different buildings are given below:

Table 4-4 Influence of Different Vulnerability Factors

Vulnerability Factors		**Increasing Vulnerability of the Building by different vulnerability factors**				
		High	**Medium**	**Low**	**N/A**	**Not known**
General	Load Path			√		
	Adjacent Building		√			
	Weak Story			√		
	Soft Story			√		
	Geometry			√		
	Vertical Discontinuity			√		
	Mass			√		
	Torsion					√
	Deterioration of Material	√				
	Cracks in Wall		√			
	Cantilever	√				
Lateral Force Resisting	Redundancy			√		
	Interfering wall		√			

System	Strong Column/Weak Beam			√		
Connection	Connectivity between different structural elements		√			
Diaphragm	Diaphragm Continuity			√		
	Plan Irregularities			√		
	Diaphragm Reinforcement at Openings				√	
Others	Pounding Effect		√			
	Nonstructural Elements		√			

4.7.1 Reinterpretation of the Building Fragility Based on Observed Vulnerability Factors

The assessment of different vulnerability factors show that the stadium falls under the average category of Reinforced Concrete Ordinary Moment Resisting Frame structure typology. The performance of stadium to different earthquake intensity is given in Table 4-5. (Refer ANNNEX II)

Table 4-5 Reinterpreted Fragility of the Structure

MMI	VI	VII	VIII	IX
Building Performance	-	DG1	DG2	DG3

Note: Refer ANNEX I for detail physical assessment.

5. NON DESTRUCTIVE TEST

Non-destructive tests are used as relatively accurate method for the assurance of the quality of the existing material such that concrete, steel bars etc. without destroying the existing features and affecting the load carrying capacity of the structure. It can be very effective when other methods of destructive tests cannot be applied for determination the properties of existing materials.

5.1 IDENTIFICATION OF TESTING LOCATIONS

After the review of drawings and pre-site visit the testing locations and numbers of testing in each block were identified and informed to the focal person of the stadium for removal of plaster covering/finishes.

5.2 SURFACE PREPARATION WORKS

Since the building's structure in a normal condition is concealed by the architectural finishes, so necessary surface preparation works i.e. removal of those finishes were required before the testing at the pre-identified locations.

Figure 5-1 Surface preparation works

5.3 TESTS

Following non destructive tests were carried out to determine the properties of existing material during the detail investigation.

5.3.1 Schmidt Hammer Test

The most common method to determine the compressive strength of concrete without destructing it is by using Schmidt hammer. The Schmidt hammer test enables us to determine the in-situ compressive strength of the concrete and assess the current strength of the concrete as it stands today.

The test was conducted on columns, beams and slab of each block – Block B, C, D, E and VIP block. The results showed that the surface of the concrete has hardened due to carbonation of concrete surface and hence obtained value is quite higher i.e., 30MPa though the estimated compressive strength of the concrete should not be more than 20 MPa.

Figure 5-2 Schmidt Hammer

Figure 5-3 Carrying out Schmidt hammer test

5.3.2 Ultrasonic Pulse Velocity Test

It involves the measurement of electronic pulses passing through concrete from a transmitting transducer to a receiving transducer. The method is based on the principle that the pulse velocity passing through the concrete is primarily dependent upon the density and elastic properties of the materials and is independent of geometry. The test is used to assess the homogeneity and quality of concrete. Any flaws or any deterioration of concrete can easily be detected.

Table 5-1 Quality indication of concrete based on velocity of wave

V (Km/s)	Quality of Concrete
>4.5	Excellent
3.5-4.5	Good
3.0-3.5	Doubtful
2.0-3.0	Poor
<2.0	Very poor

Figure 5-4 performing Ultra sonic pulse velocity

This test was performed on the columns and slabs of each block- block B, C, D, E, VIP block. The test showed that the average velocity in the concrete columns is 2918.72 m/s i.e. 2.92 Km/s which falls in the poor category of Table 5-1.

5.3.3 Cover Meter and Rebar Detector Test

Cover meter and rebar detector is used to determine the location and size of rebar and clear cover in a concrete. The basic principle of the detector is the interaction between the reinforcing bar and a low frequency magnetic field.

The test detected only 8 numbers of rebar in the columns of block B, C, D and E which is lower than the required reinforcement.

Figure 5-5 Cover meter and rebar detector test

The detail results of non destructive test are attached in ANNEX III.

6. QUANTITATIVE APPROACH OF STRUCTURAL PERFORMANCE EVALUATION

This approach includes structural analysis of a block of the stadium –non-linear static (pushover) analysis using structural analysis software named SAP2000. The material properties are considered assuming the material of full strength as if the members were newly built and thus the analysis can only be considered as preliminary and the structure may require a detail rigorous analysis considering its actual properties and accurate member sizes. Block D was taken as sample block for the analysis as it resembles most of the blocks.

6.1 INPUT DATA

6.1.1 Loads and Loading

The main types of loads to be directly considered for the design of building structure are vertical loads (dead and imposed load) and lateral loads (earthquake and wind load). As per the provision in IS 875:1987, wind load is neglected.

a) Dead Loads:

The gravity loads due to self weight of structural elements are determined considering the dimensions of elements and unit weight IS: 875 (part 1):1987. The dead load is considered the weight of structural elements including walls, finishing work and all other permanent features in the building.

b) Live Loads:

The live load considered for various usage of space office, corroder, lobbies, parking and staircase are taken as per codal provision in IS: 875 (part 2):1987. According to code the load adopted for analysis of structure are; for terrace/seating area: 5 KN/m^2 and for office space, staircase and corridor: 4 KN/m^2.

c) Earthquake Loads:

Earthquake load is calculated using Seismic coefficient of equivalent static force analysis method for zone V (Kathmandu) according to the codal provision in IS: 1893 (part 1):2002.

6.1.2 Types and Grades of Principle Member

As mentioned earlier, the grade of the concrete is unknown and so are the rebar material properties. Thus here the analysis is carried out making suitable assumptions regarding the grade of the concrete and yield strength of the rebar. The concrete used hereby for the analysis is of Grade M20, and steel is of Grade Fe 415 (Tor steel).

6.1.3 Depth of foundation

The depth of foundation is mainly governed by factors such as scour depth and nature of subsoil strata to place foundation, basement requirement and other environmental factors. As there are no rivers in the immediate vicinity of the building site, chance of scouring is absent. The analysis is carried out for only super structure; no detail investigation of foundations has been done. The analysis of foundation will be done if client wants any further detailed study.

6.1.4 Liquefaction Potential

These proposed buildings are located in high liquefaction potential area as per specification of KMC Liquefaction Potential Map 2006 (See Annex-8).

6.2 MODELING AND STRUCTURAL ANALYSIS

SAP 2000 was used as a tool for modeling and analysis of the building. SAP 2000 is the most sophisticated and user friendly series of computer programs. Creation, modification of models, execution of analysis and checking and optimization of the design can be done through this single interface. Graphical displays of the results including real time, display of time history displacements are easily produced in it.

The structure is a composite type consisting of both RCC and load bearing masonry construction. Thus the structure is analyzed here for the RCC portion only detaching the load bearing portion from the RC side. The drawings were reviewed for the approximate sizing of the structural members. Structural analysis program SAP 2000 is used for modeling and structural analysis to check the member capacity of structure.

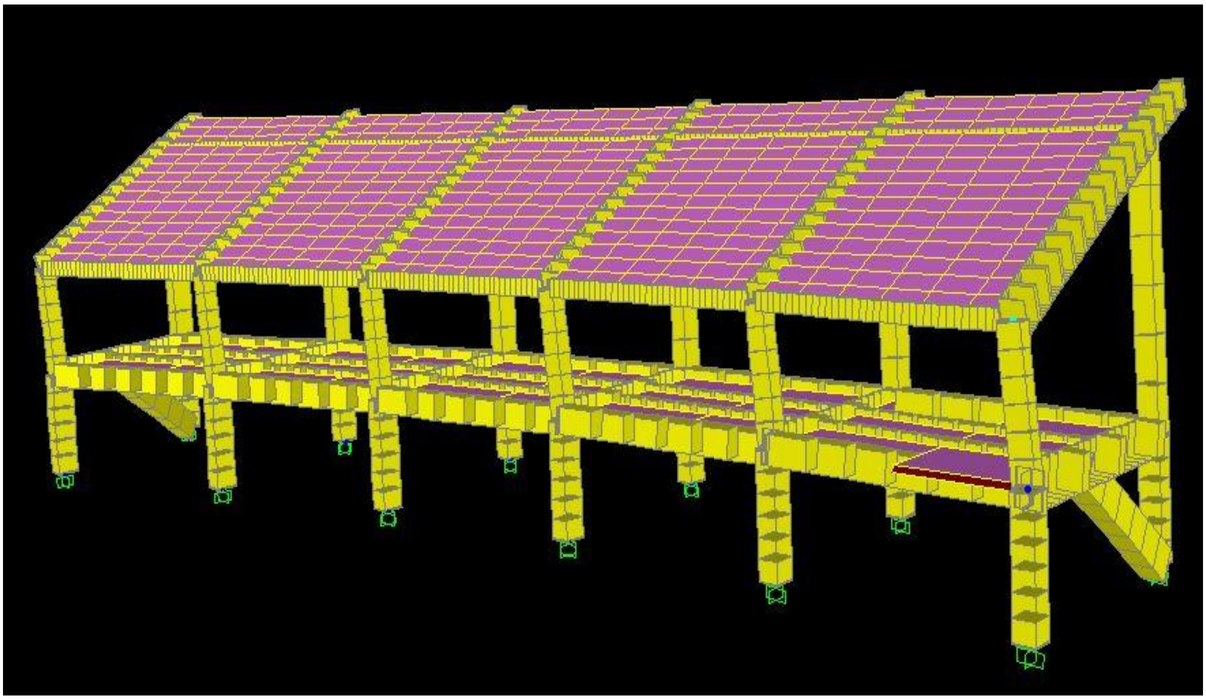


Figure 6-1: Analytical Model of D-Block

The structure was modeled as a three dimensional reinforced concrete structure to determine the required strength of the structure. The gravity (dead and live) load applied in combination with lateral load (seismic load) as recommended by IS 1893 (part1) 2002 in analysis. The pushover analysis was performed as per ATC-40 AND FEMA 356. The analysis is done based on the principle of finite element method.

6.3 RESULTS AND FINDINGS

The analysis of the finite element model shows that the structure is relatively less vulnerable in X-direction whereas it showed greater vulnerability in Y-direction at few locations.

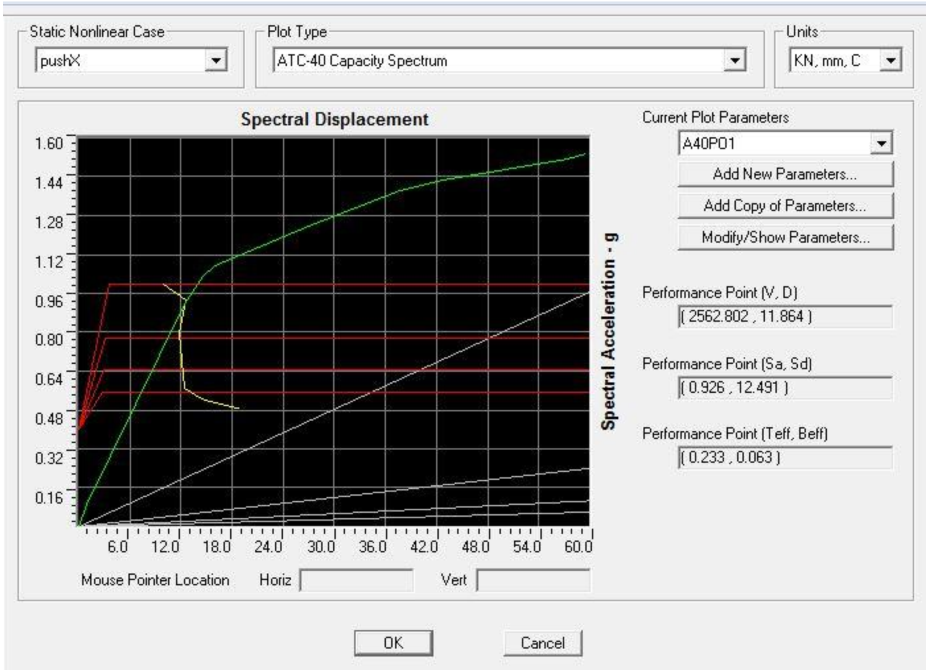


Figure 6-2: Capacity Spectrum in X-direction.

The above curve shows the capacity spectrum of the structure along X-direction which clearly shows that the performance point of the structure is when the base shear is 2562.80 KN and the maximum displacement of the structure reaches 11.96 mm.

Pushover Curve - pushX

Step	Displacemer	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	mm	KN									
0	0.465465	0.000	118	0	0	0	0	0	0	0	118
1	1.266055	222.729	116	2	0	0	0	0	0	0	118
2	11.782329	2548.168	102	16	0	0	0	0	0	0	118
3	13.066973	2777.527	96	22	0	0	0	0	0	0	118
4	14.047554	2872.507	92	26	0	0	0	0	0	0	118
5	26.963173	3456.874	85	30	3	0	0	0	0	0	118
6	36.346529	3808.569	69	37	12	0	0	0	0	0	118
7	39.356860	3859.475	66	40	12	0	0	0	0	0	118
8	47.925985	3934.211	66	39	7	6	0	0	0	0	118
9	49.161642	3951.581	65	38	9	5	0	1	0	0	118
10	49.177774	3951.366	65	38	9	5	0	1	0	0	118

Figure 6-3: Pushover curve calculation in X-direction.

The above table shows that the performance point of the structure occurs when all the hinges are in immediate occupancy zone and thus the structure does not face much threat to its safety in this direction.

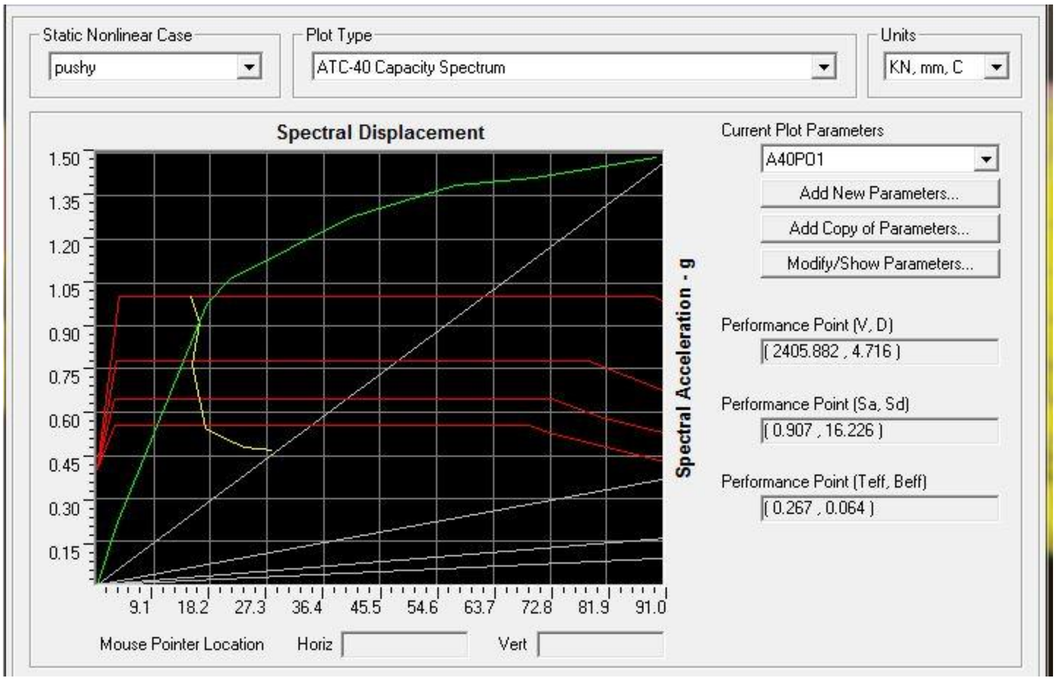


Figure 6-4: Capacity Spectrum in Y-direction.

The above curve shows the capacity spectrum of the structure along Y-direction which clearly shows that the performance point of the structure is when the base shear is 2405.88 KN and the maximum displacement of the structure reaches 16.23 mm.

Pushover Curve - pushy

Step	Displacemer	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	mm	KN									
0	0.465465	0.000	118	0	0	0	0	0	0	0	118
1	1.507132	604.996	117	1	0	0	0	0	0	0	118
2	5.050384	2593.473	80	38	0	0	0	0	0	0	118
3	5.610781	2741.854	70	48	0	0	0	0	0	0	118
4	8.678069	3179.706	59	57	2	0	0	0	0	0	118
5	11.65567	3410.700	52	51	15	0	0	0	0	0	118
6	14.95090	3480.016	48	46	24	0	0	0	0	0	118
7	18.03193	3622.402	48	39	25	5	0	1	0	0	118
8	18.03464	3622.434	48	39	25	5	0	1	0	0	118
9	18.03464	3622.434	48	39	25	5	0	1	0	0	118
10	18.035651	3622.451	48	39	25	5	0	1	0	0	118
11	18.037871	3622.429	48	39	25	5	0	1	0	0	118
12	18.03812	3622.432	48	39	25	5	0	1	0	0	118
13	18.03816	3622.433	48	39	25	5	0	1	0	0	118
14	18.038171	3622.433	48	39	25	5	0	1	0	0	118

Figure 6-5: Pushover curve calculation in Y-direction.

The above table shows that the performance point of the structure occurs when few hinges are in collapse prevention zone and thus the structure faces much threat to its safety along this direction.

The possible location of the collapse initiation in the structure is depicted in the figure below

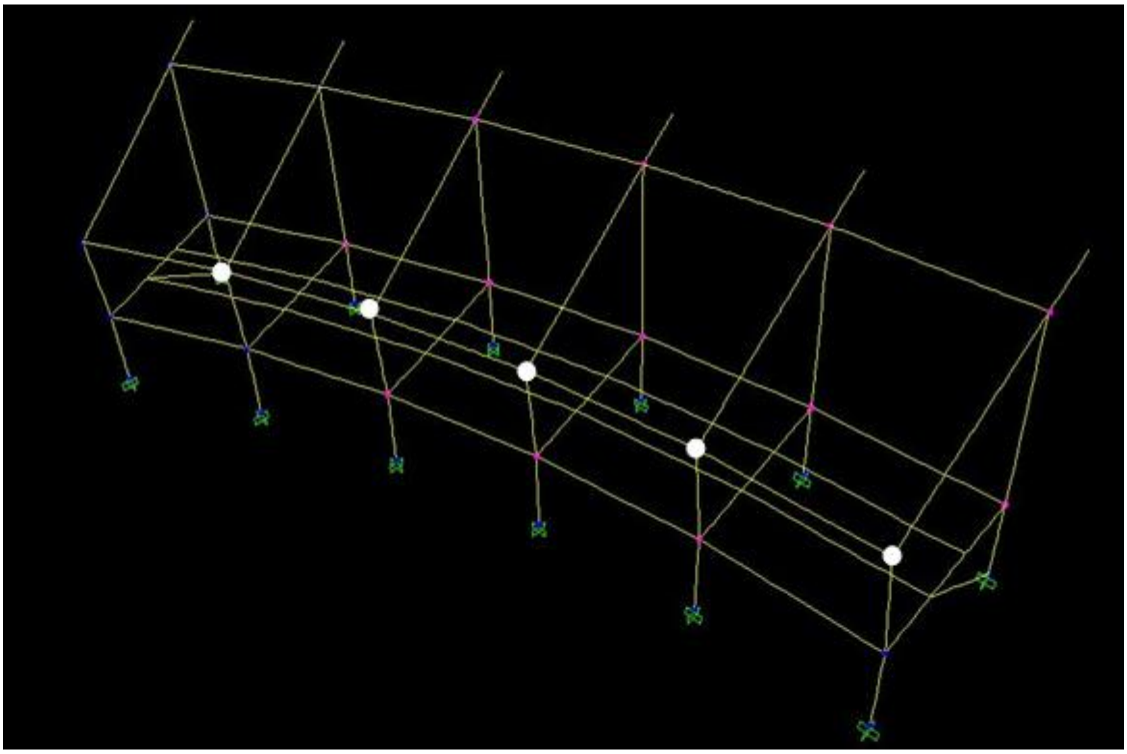


Figure 6-6: Possible Failure initiation location (indicated in white dots)

From the findings mentioned above, it can be concluded that the structure is relatively vulnerable along its length (Y-direction) than along X-direction. Thus it can be concluded that the structure needs some rehabilitation measure against earthquake. However the data used for the above analysis is preliminary and hence the structure needs detailed study of material properties and existing condition and analysis regarding the actual measures which have to be applied for its seismic strengthening.

7. FUNCTIONAL ASSESSMENT

7.1 INTRODUCTION

Apart from structural assessment, functional planning assessment was carried out against various checklists regarding the general guidelines for stadium. Despite of the stadium suffering no external or internal damages to its structural components, it may be in risk during hazards due to disruption of lifeline and infrastructure, overcrowding, lack of staffs in emergency.

7.2 CHECK FOR COMPONENTS

The individual components were then checked for the safety and for the smooth operation without creating any havoc during the crisis. The components are listed and checked as per the existing criterion as mentioned below:

7.2.1 Seating Area

The existing seating arrangements were checked with the general seating guidelines followed in various international stadiums around the world:

Width of seat : 500 mm

Total depth : 800 mm

Seat depth : 350 mm

Circulation area: 450 mm

For individual wheelchair,

Minimum width of stand = 900 mm

Minimum depth of stand = 1400 mm

7.2.2 Accommodation

- The blocks of seats should not extend back, more than twenty-three seats.
- The passing-gap in front of each row should be at least 0.30m.

- The actual size of each seat is not laid down in regulations, but at least 0.75m should be allocated between the centerlines of the rows and 45-50cm width for each person.

Table 7-1 Check for seating area/ accommodation

		Recommended	Existing	Remarks
Seating area / Accommodation	Width of seat	500 mm	510 mm	O.K.
	Total depth	800 mm	820 mm	O.K.
	Seat depth	350 mm	370 mm	O.K.
	Circulation area	450 mm	450 mm	O.K.
	Block of Seats Extended Back	23 Nos.	21 Nos.	O.K.
	Passing Gap	0.3 m	0.3 m	O.K.
	Max. Persons per Block	2500 Nos.	1200 Nos.	O.K.

7.2.3 Staircases:

The maximum width of the staircase may be calculated from the following formula:

Maximum no. of visitors x 1.25 = width of stair blocks should be required. The time for quitting is generally between 5-10 minutes.

Table 7-2 Check for staircase

		Recommended	Existing	Remarks
Staircases	Width between handrails	1.2 m	1.2 m	O.K.
	Head Room	2.1 m	2.2 m	O.K.
	Landings length	1.8 m	2.0 m	O.K.
	Riser	150- 170 mm	170 mm	O.K.
	Tread	280- 425 mm	290 mm	O.K.
	Time for Quitting	5-10 minutes	2-3 minutes	O.K.

7.2.4 Exits:

- Rows of seats must have a maximum no. of 25 seats (Fire regulations specify that a person must not have to traverse more than 14 seats to get to a passageway).

- Each block of seats must have more than one exit, while at least one exit must be provided for every 750 persons.
- Signage must clearly indicate entrances and exits.
- A wall or fence may enclose the area surrounding the stadium. It shall be at least 2.5 meters in height and shall not be easy to scale, penetrate, pull down or remove.
- Entry and exit points in the stadium and the concourse surrounding the stadium shall be designed in such a way as to facilitate the flow of persons and vehicles in and around the stadium.

Table 7-3 Check for exits

		Recommended	Existing	Remarks
Exits	Max. No. of seats in a row	25 Nos.	25 Nos.	O.K.
	No. of exit per block	2 Nos.	2 Nos.*	O.K.
	Persons per Exit	750 Nos.	700 approx.	O.K.
	Placement of Signage	Every entry and exit	Not placed	Not O.K.
	Luminance	100 Lux.	>100 lux	O.K.
	Emergency Exits	2 Nos.	2 Nos.	O.K.
	Access for Emergency/Utility Vehicle	At least 1	Two	O.K.

*Note: Blocks E and F are not provided with two exits and thus the number of persons per exit for these blocks can go considerably high and also the seating area on the natural ground slope has no practical exits because the exits over there are sealed for security purpose and may result in havoc during crisis situation.

7.2.5 Emergency gate

- The emergency exit gates shall have one door, be wide enough and remain staffed and unlocked at all times.
- It is important to provide adequate access to the pitch for any equipment and vehicles that are required in case of an emergency (police vehicles, ambulances, fire engines, etc.). It is recommended that at least one larger access point, preferably at one of the corners of the pitch, is made available for this purpose

7.2.6 Evacuation routes

- Emergency evacuation routes, one inside and one outside the stadium, must be agreed upon with the local security forces (police, stewards, fire service, first aid and emergency services). The external evacuation route shall have two lanes and be negotiable by vehicle.
- Adequate areas are required around the stadium to allow for the accommodation of spectators following an evacuation without overcrowding. The size and location of such areas should permit the free access of the police, fire and ambulance services.
- The field of play within the stadium must be accessible from at least one vehicle entry point.

Table 7-4 Check for evacuation routes

		Recommended	Existing	Remarks
Evacuation Routes	Emergency evacuation routes	one inside and one outside the stadium	one inside and one outside the stadium	O.K.
	Vehicle Entry up to field of play	Two lane road access	Two lane road access	O.K.
	Adequate space around stadium	For at least half of spectators	More than half of spectators	O.K.

7.2.7 Lighting, emergency power supply

- In the event of a power failure, there shall be emergency lighting provided by a back-up power supply.

7.2.8 Fire safety

- Preventive measures, such as the removal of sources of ignition, the provision of fire doors and the adoption of sensible precautions, especially where food is being prepared, can greatly reduce this risk.
- Fire extinguishers must be provided in areas defined by the fire service. The fire extinguishers should clearly indicate the steps to be followed for their effective use and replacement date.

- The provision of at least two subways but constructed as protected escape routes leading from the central activity space to final exits.
- At all matches, the inner areas of the stadium shall be equipped with buckets of sand and flame-retardant gloves.

Table 7-5 Check for lighting, emergency power supply and fire safety

		Recommended	Existing	Remarks
Lighting, emergency power supply & Fire safety	Emergency Lighting	Always (During Matches)	Always (During Matches)	O.K.
	Provision of Escape Routes	At least two	Two	O.K.
	Placement of Fire Extinguishers	As per Fire safety Norms	Not available	Not O.K.
	Provision of sands and fire retardant gloves during all matches	Always (During Matches)	Not available	Not O.K.

7.2.9 First aid

- The medical service shall be permanently provided with suitable rooms for the first aid treatment of spectators and any other person, other than the doping test room or the players' medical attention room.

Table 7-6 Check for first aid facilities

		Recommended	Existing	Remarks
First Aid	Availability of Primary treatment kit	Always	Always	O.K
	Provision of minor injury treatment hall (OT/ X-RAY)	Always (During Matches)	Always (During Matches)	O.K

7.3 CONCLUDING REMARKS

By examining the components individually and in groups, it can be summarized that the functional condition of the stadium is satisfactory in the current situation except for a few cases. The exits for the blocks E and F and also the passage for the seating area on the natural ground slope need to be seriously looked upon. The fire safety standards required seems to be in miserable condition. This implies for the better preparedness for the hazard situation, if ever there is any. We recommend the improvement on the situation/cases mentioned above.

8. FINDINGS AND RECOMMENDATION

8.1 FINDINGS

Apart from the general conclusions presented in the respective chapters, the specific and important conclusions are presented here in a systematic way.

- The design concepts prevailing in the present context have changed a lot since the stadium was designed and constructed. Hence the present condition of the stadium cannot be considered as safe while considering the latest design adaptation.
- The walls except at a few locations are in good condition and most of the walls are observed to be suffering from non-structural damage. But a few walls have cracks being developed at the corners where two walls meet and also at the slab-wall junction.
- Slenderness ratio of the walls is satisfactory for most of the walls except at one or two locations. The size of the opening in stadium is found to be in compliance with the opening criteria provided in NBC 109.
- Too many damp patches are found in the walls which have resulted in the spalling of the concrete.
- The joints between any of the two blocks are in vulnerable condition and the seepage problem arising from its present situation is creating adverse effects on the original strength of the concrete.
- At some places, the concrete mass found has not been properly compacted during the construction up to its desired quantity. This has resulted in the formation of loose concrete in the load- sensitive areas prone to the hazard. This type of concrete has also been verified by the non-destructive tests that were done at the site.
- No seismic bands, such as bands above lintel, were found at the site in any of the structures. The walls have not been tied down to the frame anywhere and the connection is missing. Thus the possibility of the out-of-plane failure of the walls is very high.
- The non-destructive tests carried out in the field for slabs, beams and columns showed that the concrete is in poor condition and losing its strength with the passing of time. The analytical results confirm the deterioration.

- The carbonation of the concrete along with the poor maintenance has contributed great deal in the reduction of the strength of the concrete and the rebar being used.
- The structure is being loaded with cyclic loading since its construction and it has been a considerable time since then. The reduction in the strength of the concrete due to fatigue cannot be looked upon as is shown by the analytical calculations. A simple calculation shows the strength of a M25 concrete reduces to 18 Mpa after 20 years.
- The number of exits for blocks E and F and also for the seating areas built on the natural ground slope does not satisfy the general criteria and thus can be disastrous in case of havoc.
- The fire safety preparedness does not meet the general criteria demand.
- The structural analysis of one of the block of stadium carried out with data obtained by preliminary study which shows the structure is vulnerable to earthquake.
- The location of stadium falls on liquefaction potential area.

8.2 RECOMMENDATIONS

The specific recommendations are listed as below:

- The structure needs a detail study of material properties, condition of existing structures and detailed analysis for the safety of the existing infrastructures against seismic hazards. The codes and the practices have changed much since then and a thorough check has to be performed according to the latest codes to ensure the safety against the hazards.
- The structure also needs a detailed finite element analysis which has certainly come a long way since the structure was designed. To meet the current demands, the only way to get absolutely assured with the safety issue of the current stadium's condition is to carry out a detailed seismic vulnerability analysis.
- The joints between any of the two blocks need to be sealed as early as possible so as to ensure that the current status of concrete does not suffer much deterioration. Also the damaged portion of the concrete due to the seepage problem needs to repaired and restored to its original strength as quickly as possible.
- The already damaged portion of the walls needs to be repaired and the joints sealed to prevent further seepage at the earliest.

- The provision of the seismic bands above lintel or just tying the walls with columns and beams could help a great deal for the seismic strengthening of the existing structures.
- The gates that are closed behind the seating area built on the natural ground slope need to be open especially during the matches, if not always. Else, any alternate passage for the exit needs to be defined for the spectators over there.
- The fire safety preparedness needs to be upgraded and the fire fighting team has to be in standby condition, whenever needed.

8.3 WAY FORWARD

The stadium seems to be vulnerable against seismic hazards at present according to preliminary study. However, the structure needs to go through the detailed structural analysis for the determination of hazard prone areas in specific and hence for the determination of specific measures to ensure the stability and functioning of the stadium structure against potential earthquake hazard.

There can be several components of a detailed structural analysis which needs to be carried out to ensure the seismic safety of the structure, some of them are in below and not limited to:

8.3.1 Non-destructive testing

The stadium structure needs detailed non-destructive/partial destructive testing for individual structural members so that the present condition of the existing load bearing members can be determined and accordingly the database can be prepared for the detailed structural (finite element) analysis of the existing structures and facilities.

8.3.2 Field-verification of drawings

The stadium structure has gone through some functional and geometrical changes due to some alterations in its structure. The drawings provided do not include all the dimensions so that the finite element modeling is not possible. Thus the provided drawings have to be verified in the field and possible alterations in the drawings provided have to be taken note of and proceed accordingly.

Also, the proper dimensions of the structural members are missing which finite element modeling requires. Thus the members have to measured properly once again for the detailed finite element modeling.

8.3.3 Finite element modeling and analysis

The structure will have to be analyzed including micro-details like also incorporating the changes in the material properties and other geometrical alterations, if any.

8.3.4 Retrofit design

After the structure is analyzed properly with the latest modifications in the geometry and including material behavior in the present context, the appropriate and specific retrofit measures can be developed keeping in mind the future possible earthquakes. The design will be done as per the retrofit guidelines accepted worldwide. It will ensure to bring down the seismic hazard to an acceptable level after the measures are implemented.

8.3.5 Detailed cost estimate and work schedule preparation

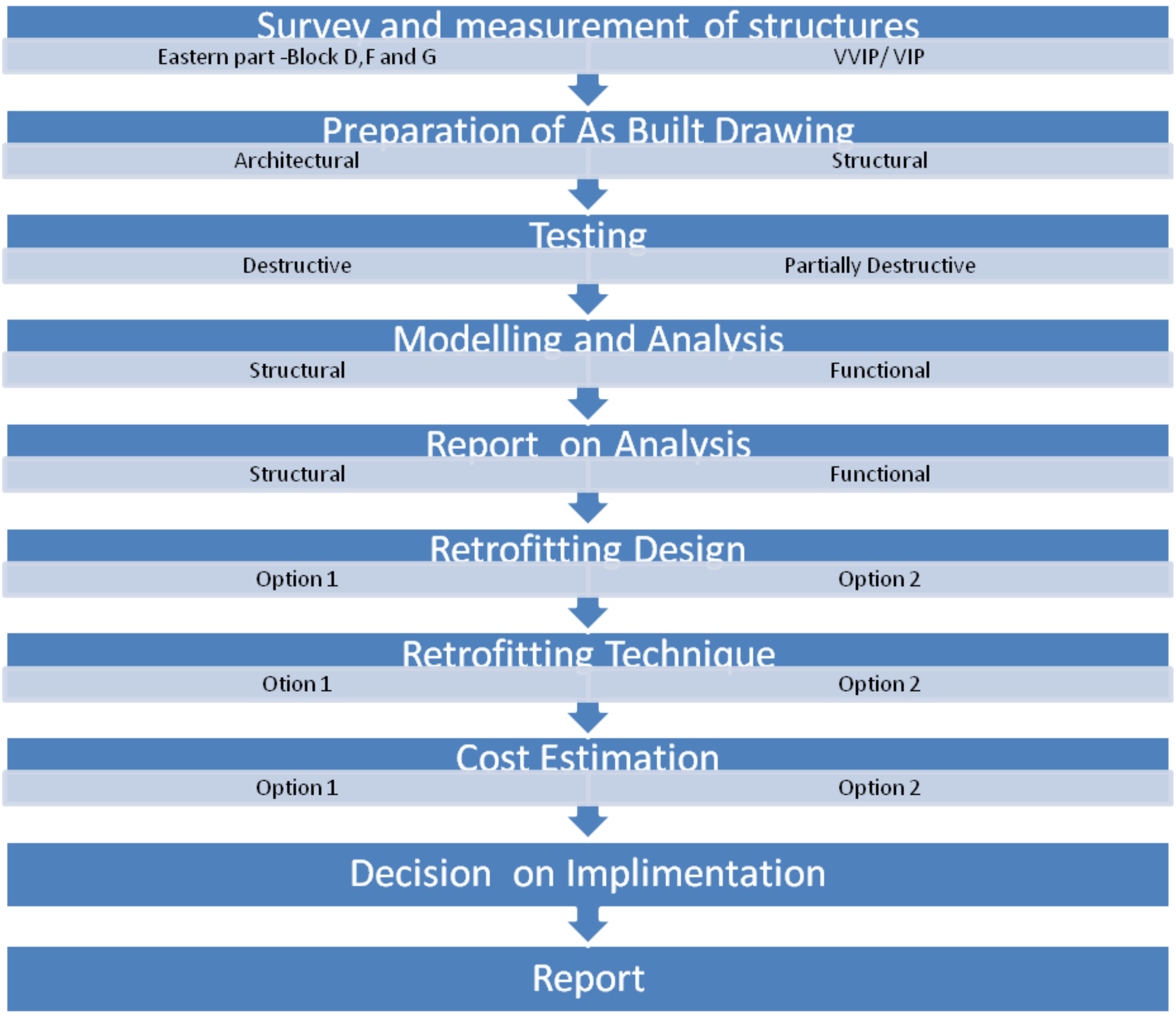
Once the specific retrofit measures are developed, the manpower and other resources that have to involved in the implementation can be computed. Hence the cost involved and also the time required for the implementation of the measures can be worked out.

Detail structural/Functional analysis and Retrofitting Design

As per the preliminary study report on vulnerability assessment, the stadium seems to be vulnerable against seismic hazard. However, the structure needs to go through detailed structural analysis for the determination of hazard prone areas in specific. Hence for the determination of specific measures to ensure the stability and functioning of the stadium structure against potential earthquake hazard, detail vulnerability assessment of the stadium is required.

There can be several components of a detailed structural analysis which needs to be carried out to ensure the seismic safety of the structure. The major activities are presented in the following flow diagram;

Flow Diagram



Estimated Budget for Detail Structural / Functional Analysis and Retrofitting Design

Activities		Unit	Quantity	Rate	Amount
Survey and Measurement		Job			100000
Preparation of as built drawing		Job			200000
Testing	Compressive strength	No	100	2000	200000
	Rebar location	No	100	3000	300000
	Core test	No	20	5000	100000
	Corrosion test	No	30	3000	90000
Modeling and analysis	Structural	Job			600000
	Functional	Job			100000
Analysis Report		Job			100000
Retrofitting Design and techniques		Job			400000
Cost Estimation/ specification		Job			150000
Final Report		Job			200000
Grand Total					**2540000**

(In words; NRs. Two million Five hundred Forty Thousand only)

Note:

- The estimated budget is developed based on CoRD's past experience.
- Geotechnical Investigation cost is not included as done previously for construction of float light and report available
- Cost for opening of foundation for investigation is not included

REFERENCES

- The Study on Earthquake Disaster Mitigation in the Kathmandu Valley, JICA and MOH, 2002
- Risk Sensitive Land Use Plan - Kathmandu Metropolitan City, EMI, 2010
- Structural Analysis Program, SAP 2000
- IS 456: 2000 Indian Standard Plain and Reinforced Concrete - Code of Practice, 4th. revision
- IS 1893 (Part 1): 2002 Indian Standard Criteria for Earthquake Resistant Design of Structures Part 1,
- IS 875 (Part 1): 1989 Indian Standard Code of Practice for Design load - dead load other than Earthquake (2nd revision)
- IS 875 (Part 2): 1989 Indian Standard Code of Practice for Design load - imposed load other than Earthquake (2nd revision)
- IS 13920: 1993 Indian Standard Code of Practice Ductile Detailing of Reinforced Concrete Structures subjected to Seismic forces
- FEMA 310, "Handbook for the Seismic Evaluation of Buildings"
- FEMA 154, “ Rapid Visual Screening of Buildings for Potential
- “Liquefaction susceptibility of Kathmandu valley”, Department of Mines and Geology of Nepal

ANNEX-I
(INFORMATION COLLECTION SHEET)

Structural Information Collection Sheet

for

Qualitative Seismic Vulnerability Assessment of

Dasarath Rangasala

Date of Visit: 2013-07-01

Project: Seismic Vulnerability Assessment of Dasarath Rangasala

Owner: National Sports Council

Location: Tripureswor, Kathmandu

Name of Assessment team:

1. Dinesh K Gupta
2. Sudeep K.C.
3. ……………………………………………………………
4. ……………………………………………………………

1. General Information

1.1 Project Name: **Seismic Vulnerability Assessment of Dasarath Rangasala**

1.2 Year of Construction: **1961 A.D.**

1.3 Drawings Available:

1. Architectural Drawing	√
2. Structural Drawing	X

1.4Building Designer and Supervision Info:

1. Design Report Available	X
2. Year of Design	1961
3. Code Referred for Design	N/A

Code	No. of times it has been revised	What has been revised?	Is it complaint with the existing design of the Stadium?	
			Yes	No
Indian Standard	SIX	General provisions including earthquake calculations		√
British	SEVERAL	General		√

Standard		provisions		
American Standard	SEVERAL	General provisions		√
Other				

1.5 Any alterations from provided drawing? If yes, list the changes.

No major changes have found until now
....
....
....
....

1.6Type of Structure:

S.N.	Types	Tick
1	Load Bearing Masonry	
2	RCC Frame	
3	Composite(RC + Load bearing)	√
4	Other.	

2. Existing Site Condition

2.1 Local Hazard

1	Existing site is prone to landslide or suffered from landslide in the past or continuation of landslide at present.	No

2	Existing site is on loose soil subjected to settlement or there are sign of settlement in past.	No
3	Existing site is in rock fall area or there are signs of rock fall in the past.	
4	Existing site is in landfill site.	
5	If any other	
6	No local hazard	√

2.2Terrain Type

1. Plain	√	2. Gentle Slope Land	
3. Steep Slope Land		4. Terrace Land	

3. General Information about Structure

3.1.Configuration of block in plan

1. Square		2. Rectangular (L<=3B)	Block I
3. T- Shaped		4. Narrow Rectangular (L>3B)	VIP Block, Blocks F,G,H

5. L- Shaped		6. U- Shaped	
7. E- Shaped		H- Shaped	
8. Describe if any other	Circular arc (segment type)- A, B, C, D, E		

3.2.Shape of block in elevation

1. Not Stepped		2. Steeped near the centre		3. Steeped near the End	√

3.3.Roof Shape

1. Flat Roof		2. Lean to Roof	
3. Stepped Roof		4. Arched Roof	
5. Cantilever Roof	√		

3.4.No. of Storey

1		2	√	3		4		5	

3.5.Open ground storey

Yes		No	√

3.6.Pounding possibility

Yes	√	No	

3.7.Cantilever with wall

1. None		2. One Side	√
3. Two opposite Side		4. Two Adjacent Side	

3.8.Position of the block i.e. attached with other blocks

1. None		2. One Side	
3. Two opposite Side	√	4. Two Adjacent Side	

4. General Information about Stands

4.1.Configuration of blocks in plan

Same as above mentioned.

1. Square		2. Rectangular (L<=3B)	
3. T- Shaped		4. Narrow Rectangular (L>3B)	
5. L- Shaped		6. U- Shaped	
7. E- Shaped		8. Curved Shape	
9. Describe if any other……………………………………………			

4.2.Shape of block in elevation

1. Not Stepped		2. Steeped near the centre		3. Steeped near the End	

4. Presence of Cantilever portion	For VIP Block only

4.3.Roof Shape

1. Flat Roof	√	2. Lean to Roof	
3. Couple Roof		4. Hipped Roof	
5. Arched Roof		6. Cantilever Roof	

4.4.Seating Area Information

1. Presence of Cantilever Portion	√	2. Open framed Structure	
3. Closed frame structure		3.1 Presence of Masonry walls	For all other blocks
		3.2 Presence of RC Shear walls	

4.5.Seating Area Roof Material

1. CGI sheets		2. Fiber sheets		3. RCC roof	√	4. Other	

4.6.Seating Area Roof Support

1. RC Framed		2. Cantilever	√
3. Dome shaped		4. Others	

4.7. Location of areas prone to excessive vibration during dynamic loading

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5. Physical Defects

Building Component	Defect	Remarks
Wall	Horizontal Crack	√
	Vertical Crack	√
	Diagonal Crack	
	Out of plumb wall	
	Crack at wall joint	√
	Damp patches	√
	Damaged plaster	√
	Crack at wall roof junction	√

Column	Vertical crack (> 1mm)	
	Out of plumb	
	Exposure of reinforcing rod	
Beam	Cracks at mid span or diagonal crack	
	Spalling of concrete cover	
	Exposure of reinforcing rod	
Ceiling	Damp patch	√
	Water seepage	√
	Spalling of concrete	√
	Exposure of reinforcing rod	√
Differential settlement (if any)		

6. Vulnerability Factors Identification Checklist (FEMA 310)

6.1. Building System

C NC N/A NK LOAD PATH: The structure shall contain at least one rational and complete load path for seismic forces from any horizontal direction so hat they can transfer all inertial forces in the building to the foundation.

C NC N/A NK REDUNDANCY: The number of lines of vertical lateral load resisting elements in each principle direction shall be greater than or equal to 2.

<u>C</u> NC N/A NK GEOMETRY: No change in the horizontal dimension of lateral force resisting system of more than 50% in a storey relative to adjacent stories, excluding penthouses and mezzanine floors, should be made.

C <u>NC</u> N/A NK WEAK STORY: The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent story.

C <u>NC</u> N/A NK SOFT STORY: The stiffness of vertical lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent story or less than 70% of the average stiffness of the three storey above.

<u>C</u> NC N/A NK VERTICAL DISCONTINUITIES: All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

C <u>NC</u> N/A NK MASS: There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouse, and mezzanine floors need not be considered.

C <u>NC</u> N/A NK TORSION: The estimated distance between the storey center of mass and the storey centre of stiffness shall be less than 30% of the building dimension at right angles to the direction of loading considered.

C <u>NC</u> N/A NK ADJACENT BUILDINGS: The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4 % of the height of the shorter building, expect for buildings that are of the same height with floors located at the same levels.

C <u>NC</u> N/A NK FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.

C <u>NC</u> N/A NK SHORT COLUMNS: The reduced height of a columns due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, infill wall, etc. or 50% of the nominal height of the typical columns in that storey.

C NC N/A NK DETERIORATION OF CONCRETE: There should be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.

C NC N/A NK CRACKS IN BOUNDARY COLUMNS: There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry *infills*.

C NC N/A NK MASONRY UNITS: There shall be no visible deterioration of masonry units.

C NC N/A NK MASONRY JOINTS: The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.

C NC N/A NK CRACKS IN INFILL WALLS: There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3mm, or have out of plane offsets in the bed joint greater than 3 mm.

6.2. Lateral Load Resisting System

C NC N/A NK SHEAR STRESS IN RC FRAME COLUMNS: The average shear stress in concrete columns tcol , computed in accordance with 6.5.1 of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings shall be lesser of 0.4MPa and 0.10 √fck.

C NC N/A NK SHEAR STRESS IN SHEAR WALLS: Average shear stress in concrete and masonry shear walls, Wall shall be calculated as per 6.5.2 of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings. For concrete shear walls, tWall shall be less than 0.4 MPa . For unreinforced masonry load bearing wall building wall buildings, the average shear stress, Wall shall be less than 0.10 MPa.

C NC N/A NK SHEAR STRESS CHECK FOR RC MASONRY INFILL WALLS: The shear stress in the reinforced masonry shear walls be less than 0.3 MPa and the shear stress in the unreinforced masonry shear walls shall be less than 0.1 MPa.

C NC N/A NK AXIAL STRESS IN MOMENT FRAMES: The maximum compressive axial stress in the columns of moments frames at base due to overturing forces alone (Fo) as calculated using 6.5.4

equation of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings shall be less than 0.25fck.

C NC N/A NK NO SHEAR FAILURES: Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provision of IS: 13920 for shear design of beams and columns.

C NC N/A NK CONCRETE COLUMNS: All concrete columns shall be anchored into

the foundation.

C NC N/A NK STRONG COLUMN/WEAK BEAM: The sum of the moments of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams at each frame joint.

C NC N/A NK BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.

C NC N/A NK COLUMNS BAR SPLICES: Lap splices shall be located only in the central half of the member length. It should be proportions as a tension splice. Hoops shall be located over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50% of the bars shall preferably be spliced at one section. If more than 50 % of the bars are spliced at one section, the lap length shall be $1.3L_d$ where L_d is the development length of bar in tension as per IS 456:2000

C NC N/A NK BEAM BAR SPLICES: Longitudinal bars shall be spliced only if hoops are located over the entire splice length, at a spacing not exceeding 150mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be located (a)

within a joint, (b) within a distance of 2d from joint face, and (c) within a quarter length of the member where flexural yielding may occur under the effect of earthquake forces. Not more than 50% of the bars shall be spliced at one section.

C NC N/A NK COLUMN TIE SPACING: The parallel legs of rectangular hoop shall be spaced not more than 300mm centre to centre. If the length of any side of the hoop exceeds 300mm, the provision of a crosstie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.

C NC N/A <u>NK</u> STIRRUP SPACING: The spacing of stirrups over a length of 2d at either end of a beam shall not exceed (a) d/4, or (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. In case of beams vertical hoops at the same spacing as above shall also be located over a length equal to 2d on either side of a section where flexural yielding side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding d/2.

C NC N/A <u>NK</u> JOINT REINFORCING: Beam- column joints shall have ties spaced at

or less than 150 mm.

C NC N/A <u>NK</u> STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall

preferably be anchored into the member cores with hooks of 1350.

C NC N/A <u>NK</u> JOINT ECCENTRICITY: There shall be no eccentricities larger than

20% of the smallest column plan dimension between girder and column centerlines. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A <u>NK</u> WALL CONNECTIONS: All infill walls shall have a positive connection to the frame to resist out-of-plane forces.

C NC N/A <u>NK</u> INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

6.3. Diaphragms

<u>C</u> NC N/A NK DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints.

C NC N/A NK PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A NK DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragms openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only.

6.4.Geologic Site

C NC N/A NK AREA HISTORY: Evidence of history of landslides, mud slides, soil settlement, sinkholes, construction on fill, or buried on or at sites in the area are not anticipated.

C NC N/A NK LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not xist in the foundation soils.

C NC N/A NK SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure.

7. Repair and Maintenance History

What has been repaired?	When?	Type of repair	
		Structural	Non Structural
Simple repair works and maintenance of whole structure	1999 A.D.		√

8. Independent check for components

8.1. Foundation Types

Not known.

Isolated Footing		Combined Footing		Raft Footing	

Frame Elements and their Condition

S.N.	Frame Elements	Size and Shape	Size and No. of reinforcement	Condition	Remarks
1	DPC Beam				
2	Column	Rectangular	Max 8 nos.	good	
3	Beam	Rectangular	NK(Not Known)		

4	Others if any				

8.2.Infill Walls

S.n.	Wall material	1st Storey		2nd Storey		3rd Storey		4th Storey		5th Storey	
		Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
1	Stone in mud mortar										
2	Stone in cement mortar										
3	Brick in mud mortar										
4	Brick in cement mortar	√	√	√	√	√	√	√	√	√	√
5	Hollow cement block in cement mortar										
6	Solid cement block in cement mortar										
7	Describe if any other										

8.3.To check whether the walls are tied with frame or not:

S.N.	Location	Remarks
1	At Sill Level	NO
2	At Lintel Level	NO

3	Other Space	

8.4.Floor

S.N.	Floor Material	First Floor		Second Floor	
		Floor Structure	Thickness	Floor Structure	Thickness
1	R.C.C.	FLAT	125 mm	FLAT	125 mm
2					
3					

8.5.Roofs

S.N.	Roof Material	Thickness	Remarks
1	R.C.C.	125mm	
2	C.G.I. Sheet		
3	Others		

Possibility of Short-Column Effect:
The columns have been properly tied at the joint level with beam so that the possibility of short column effect can be ruled out.
…………………………………………………… ……
Possibility of Soft-Storey:
The possibility of short column doesn’t seem to be in a pronounced way as the frames have been covered with infill walls of proper slenderness and stiffness so that the variation in stiffness doesn’t alter a great way

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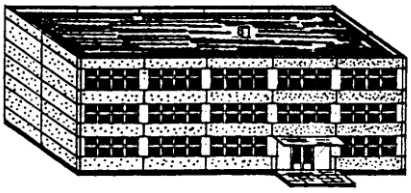
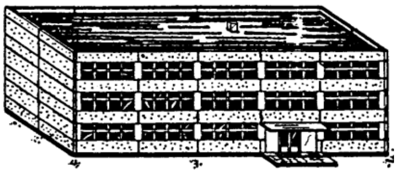
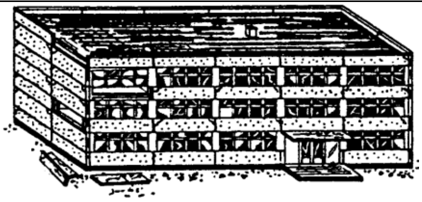
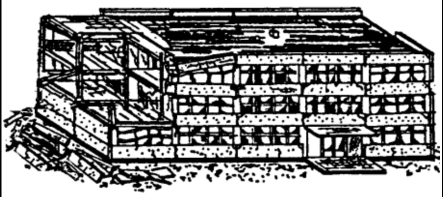
ANNEX-II

(Classification of Damage to Buildings)

Classification from European Macro-seismic Scale (EMS 98)

Classification of damage to Masonry Buildings

Classification of damage to buildings of reinforced concrete	
	**Grade 1: Negligible to slight damage** **(no structural damage,** **slight non-structural damage)** Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.
	**Grade 2: Moderate damage** **(slight structural damage,** **moderate non-structural damage)** Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.
	**Grade 3: Substantial to heavy damage** **(moderate structural damage,** **heavy non-structural damage)** Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of conc rete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.
	**Grade 4: Very heavy damage** **(heavy structural damage,** **very heavy non-structural damage)** Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.
	**Grade 5: Destruction** **(very heavy structural damage)** Collapse of ground floor or parts (e. g. wings) of buildings.

ANNEX-III

(Non Destructive Test)