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RELIABILITY ASSESMENT OF RCC FRAMES VERTICALLY IRREGULAR BUILDINGS

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ABSTRACT

In the current study, vulnerability and reliability analyzes were used to investigate performance of typical OGS buildings designedconsidering hierarchical structures with different scale factors and different geometric configurations. As intensity metric, the crucial inter-storey drift is taken into account. The result of study is shows that for 6, 8 & ten storied OGS frames at various performance levels, such as IO, LS, and CP, the likelihood of failure and reliability indices for all frames at PGA of 1.05g are determined. In contrast to full in filled frames (FF), which are able to achieve the intended dependability at all performance levels, bare frames (BF) are unable to do so. This is according to ISO 2394, 1998.

Keyword: *Open ground storey (OGS); Irregular buildings, Reliability analysis, Performance levels* etc.

INTRODUCTION

The inter-storey drift capacity corresponding to a given performance level will most likely be exceeded by maximum inter-storey drift in frames if subjected to an earthquakeof a certain severity in terms of effective PGA, according to fragility curves thus far developed. The fragilitycurves must be paired with the seismic hazard curve in study zone in order to calculate reliability, which is inversely related to the likelihood of failure, and actual probability of failure. The seismicity of particular location for which structure has been constructed should be accurately depicted by the danger curve. The Manipur region's danger curves, which are in seismic zone V, were chosen for the current study. A building was also developed. Hazard curve for a location where a 1.05g earthquake would have a 2500-year return time or a 2% chance of exceeding it in 50 years. The dependability index is determined by taking the regular normal distribution and reversing it. The Target Reliability Indices requirement for each performance level (consequences of failure) for each relative cost of measures is recommended by ISO 2394 (1988). To evaluate dependability, the current study has established target reliability values based on ISO 2394: 1988.

No design advice, specifically for OGS frames, were made for vertical irregularity in preceding IS 1893 specification. Nevertheless, the 2002 revision came about as a result of the Bhuj earthquake. A new design suggestion for OGS structures was added into the most recent edition of IS 1893 (2002).

The soft storey's columns & beams must be built for 2.5 times storey shear and moments computed under the seismic load of bare frame type structures, according to clause 7.10.3 (a). Kanitkar & Kanitkar (2001), Kaushik, & Subramanian (2004) all study the magnification factor (MF) 2.5.. The "strong beam - weak column" scenario is likely to emerge from using the Magnification Factor MF 2.5 in beam design.

As suggested by the preceding phrase, it is not necessary to design the soft-story beams to additionally account for higher storey shears. The pressure on the columns will rise even more as a result of the strengthened beams, which will also prevent plastic deformation of the beams. Due to the concentration of substantial reinforcement in the column, these suggestions have encountered considerable opposition in design & construction practice.

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The following five categories of anomalies for structures are given out by the IS 1893 (2002) code:

1. A story is considered to be stiffness irregular if its lateral stiffness is less than either 70% of story above it or 80% of mean inflexibility of the 3 levels more than it.
2. When any narrative's effective mass exceeds 150 percent of the effective mass of a story next to it, weight (mass) irregularity is said to occur.
3. When flat measurement of lateral force-resisting scheme in any story exceeds 130 percent of to in a neighboring story, vertical geometric irregularity is said to be present.
4. When an in-plane offset of lateral-force-resisting elements is more than length of those elements or when there is a decrease in the stiffness of resisting element in narrative below, in-plane discontinuity - in vertical lateral-force-resisting elements is said to occur.
5. Discontinuity in Capacity - A weak tale is one whose lateral strength is less than 80% of the narrative's lateral strength in the story mentioned above. The sum of the lateral strengths of all seismically resistant components that share the story shear in the direction under consideration is the story lateral strength.

OPEN GROUND STOREY (OGS) BUILDING

Stiffness abruptly drops, which is known as stiffness irregularity, as a result of the infill walls being absent at the lowest floor but present at all other storeys. The ground story columns detect the base shear. Plastic hinges are created as the shear force increases, which also increases bending moment & curvature. This may lead to higher inter-storey drift creation at the ground level & is accentuated by P-effect. A single block will travel with the upper shop. Soft story collapse is the name given to this sort of collapse. This sort of construction is thought to be the most susceptible due to the decrease in stiffness at the ground level.

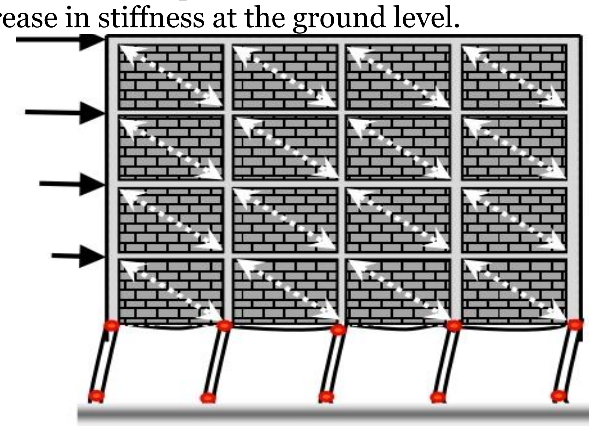

Fig [1] Lateral load OGS frame behavior

AIM OF THESTUDY

The main aim of this study is to carry out a reliability study & determine the values of the reliability indices for each building frame.

LITERATURE REVIEW

Afarani & Nicknam (2012) used incremental dynamic analysis to observe the behaviour of vertically irregular buildingunder seismic stresses. They are dealing with an eight story conventional structure that has two bays that are 4 m wide in ydirection & four bays that are 3 m wide in the x direction,with a storeyheight of 3 m. Incremental dynamic analysis is used to compute maximum inter-story drift ratios and first mode spectral acceleration, & the IDAcurved is plotted to determine collapse locations. The building's examination is concentrated on the structures' ability to prevent collapse. By applying the Cumulative Distribution Function & the collapse points of lognormal distribution, fragility curves are produced.

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Samoah (2012) studied non-ductile reinforced concrete (RC)frame structures' fragility behaviour in low- to medium-seismic regions, preferring Accra, the capital of Ghana in West Africa. Inelastic pushover analysis is used to examine the structural strength of the structures, while inelastic time history analysis is used to evaluate seismic demand.

Marano (2011) generated fragility curves based on categorization & structures offered by Hazus database. Buildings of 2 story & 5 storey kind are taken into consideration, as are those with low seismic design & those with medium seismic design. For the fragility analysis, a damage index based on displacement is used.

Sarkar (2010) examined the irregularity in stepped framed buildings. Seven numbers of buildings with varied heights are also includedwithout taking step into account. 78 building frames with uniform number and bay width of 4 & 6 metres, respectively, are taken into consideration. 50 modes are concentrated for four distinct building situations.

Erberik (2008) looked at low- & mid-rise reinforced concrete (RC) structures that had been damaged by two severe earthquakes in the Marmara area of Turkey in 1999. Buildings of two to six floors have been taken into consideration. Buildings of two & three floors are called low-rise (LR) in the study, & those with four to six stories are referred to as mid-rise (MR).

According to Davis & Menon's (2004) research, an OGS building's structural force distribution is dramatically altered by the inclusion of masonry infill panels. In order to analyse how the behaviour & performance of the structures changed as number of storeys & bays as well as storey heights increased, a variety of architectural case studies were taken into consideration.

Kim & Shinozuka (2004) investigated fragility study of two sample bridges that had been modified with steel jacketed bridge columns. The first of the two bridges had three spans that were each 34 metres long, 2 half shells made of rolled steel plate, & an RCdeck slab that was 10 metres wide. It was supported by two pairs of circular columns, each with three columns that were 0.8 metres in diameter & abutments.

Tantala & Deodatis (2002) took into consideration a building with a three-bay, 25 storey reinforced concrete moment resistant frame. Over a wide variety of groundmotion intensities,they have produced fragility curves. They have employed stochastic processes to model temporal histories.

EISMIC RELIABILITY ANALYSIS FOR VARIOUS HAZARD SCENARIOS

The danger curve of the area for which the structure is constructed must be integrated with the fragility curves created in the preceding Chapter. The yearly probabilities of particular amounts of earthquake motion are used to represent the seismic hazard P [A = a]. The danger curve created for Manipur is used in this study. By taking into account a succession of (increasinglsevere) limit states, LSi, formula may be used to compute limit state probabilities:

$$P\ \{LS_i\} = \sum_a \{LS_i \mid A = a\} P\{A=a\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \{1\}$$

According to Cornell et al. (2002), fragility FR(x) & derivative of seismic hazard curve GA(x) may be convolved to remove the acceleration-related conditioning and get a point estimate of limit state probability for state eq 1.

$$P\{LS_i\} = \int F_R\ \{x\}\ \frac{dGA}{dx} dx \ldots\ldots\ldots\ldots\ldots\ldots \{2\}$$

Eq. 2 is numerically integrated to determine the likelihood of failure. Figure 1 provides a pictorial explanation of numerical integration. The fragility and danger curves are separated into narrow strips that run parallel to the vertical axis. For each strip, the ordinate of the fragility curve is multiplied by slope of hazard curve, & sum of all strips is then used to calculate likelihood offailure.

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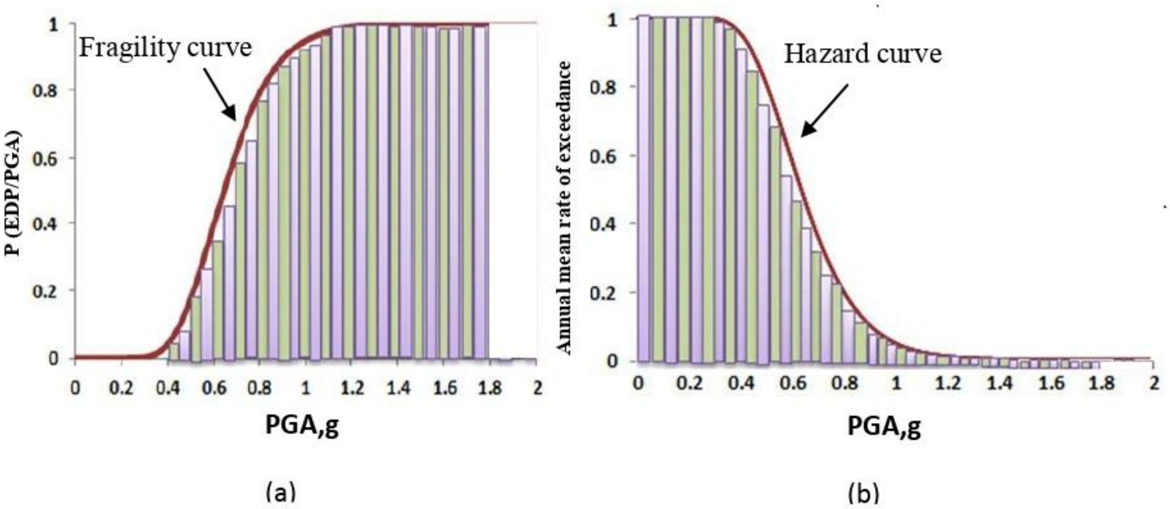

Fig [1] (a) fragility curve & (b) hazard curvefor probability of failure

For probability estimate to be useful, parameters at the fragility-hazard interface must be dimensionally consistent. Thefollowing common Equation may be used to get the reliability index for the associated likelihood of failure.

$$B = -\Phi^{-1}\{pf\}\ldots\ldots\{3\}$$

ANALYSIS OF SEISMIC HAZARDS

A complementary cumulative distribution function (CCDF), as per, is used to visualise seismic risk at a construction site. The annual frequency for exercise intensity x above a certain level is hazard function. Basic seismic hazard analysis shows that there is log-linear relationship b/w annual maximum seismic ground acceleration or spectral acceleration and probability, GA(a), that specifies values of acceleration are exceeded, as reference, at moderate to large values of ground acceleration. This connection means that the equation that follows describes

$$G_A\{x\} = 1\text{-}\exp\{-\{x/u\}^{-k}\}\ldots\ldots\{4\}$$

The parameters of distribution are u and k. The danger curve's slope is determined by the parameter k, which is connected to the yearly maximum peak acceleration's COV.

Pallav et al. (2011) provide an approach for evaluation of seismic risk ofbuilding structures. The hazard analysis estimates the likelihood that a specific amount of ground motion will be exceeded in 50 years. The risk is determined by factors such as the frequency and magnitude of earthquakes, the nature of the rocks and sediments that earthquake waves pass through, etc.

The probabilistic seismic hazard curve of Manipur is chosen for current study since it is located in the seismically active area of North East India. Pallav et al. (2011) produced the curve in question. Figure 2 depicts the danger curve for the Manipur area.

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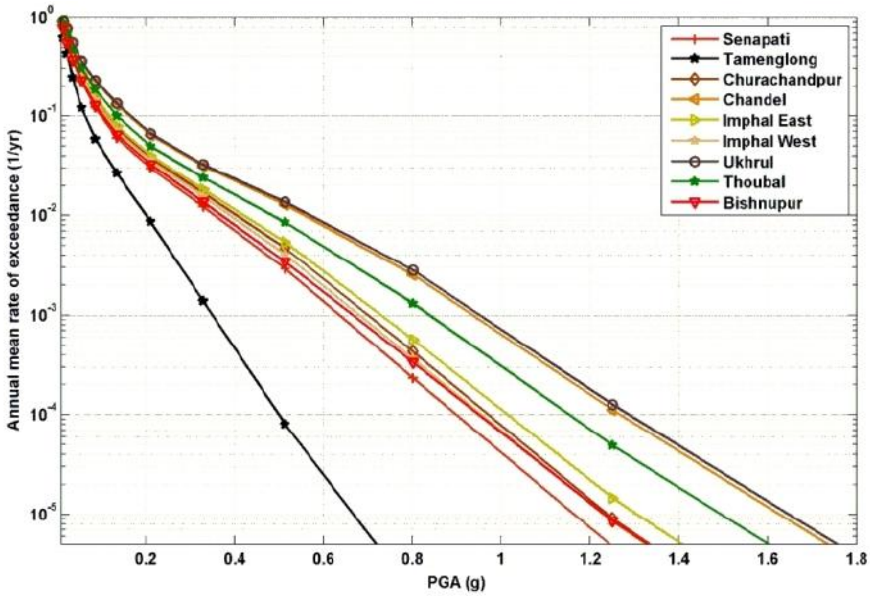

Fig [2] The danger curve for the Manipur area.

Fig 2 (Pallav et al., 2011) depicts the risk curves for the various regions in the Manipur area. The danger curve for the Ukhrul site is taken from the graph in Fig 2 and displayed as shown in Fig 3.

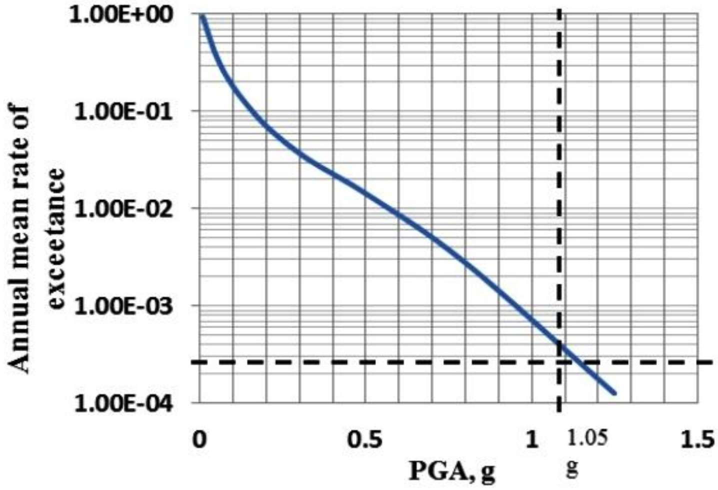

Fig 3 A failure probability hazard curve's numerical integration

USE OF THE RELIABILITY INDICES IN THE EVALUATION

As per the method described in the preceding section, the dependability index is computed from the fragility curves. Each PGA's dependability index is determined, resulting in reliability indices specific to each PGA. Target Reliability Indices in accordance with ISO 2394 (1998) are utilized in the current investigation and are displayed in Table 1. The target reliability requirement (consequences of failure) is shown in this table for each performance level. By comparing reliability indices acquired for each building with the goal reliability indices corresponding to moderate degree of failure effects, each building's performance is evaluated. The goal dependability index is chosen as 3.8 in order to evaluate how well the structures perform in preventing collapses.

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Table 1 According to IS 2394 (1998), the target dependability index

Relative Costof Measure	Consequences offailure		
	Some IO	Moderate LS	Great CP
High	1.49	2.29	3.10
Moderate	2.49	3.102	3.79
Low	3.101	3.79	4.29

For 8-story, 6-bay frames, the variation of the dependability index (β) with the parameter, PGA is depicted in Fig 4 and 5 for OGS frames & stepped irregular frames, respectively. The dependability index is shown to decline as the PGA rises. The ISO 2394 (1998) target dependability for a moderately damaged structure is indicated in Fig 4 as being 3.8. According to the Manipur region's hazard curve (shown in Fig 4 & 5), the PGA with a 2% chance of occurring in 50 years is found to be 1.05g. Reliability indices for the bare frame building (built in accordance with Indian Standards) at PGA of 1.05g are 3.27, as shown in Fig 4. In the Manipur location, it is discovered that the bare frame failed to achieve target reliability of 3.8 at PGA of 1.05g, which corresponds to a 2 percent chance of occurrence in 50years. At PGA of 1.05g, OGS frames (modelled with infill wall stiffness and strength) attained a dependability of greater than goal reliability (3.8). The fluctuation in reliability indices for different PGAs for 8-story, six-bay vertical frames with stepped topologies & infill wall layouts is shown in Fig 5. At a PGA of 1.05g, all bare frames fail to meet target reliability requirement. Even in stepped vertically uneven constructions, the presence of infill walls is crucial to achieving the desired dependability.

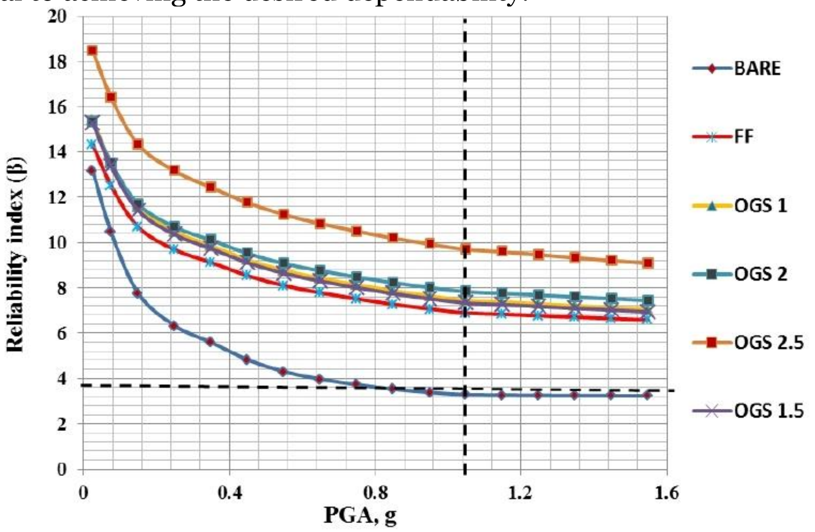

Fig 4 Reliability 8-story, 6-bay OGSframes with curves for CPperformance levels

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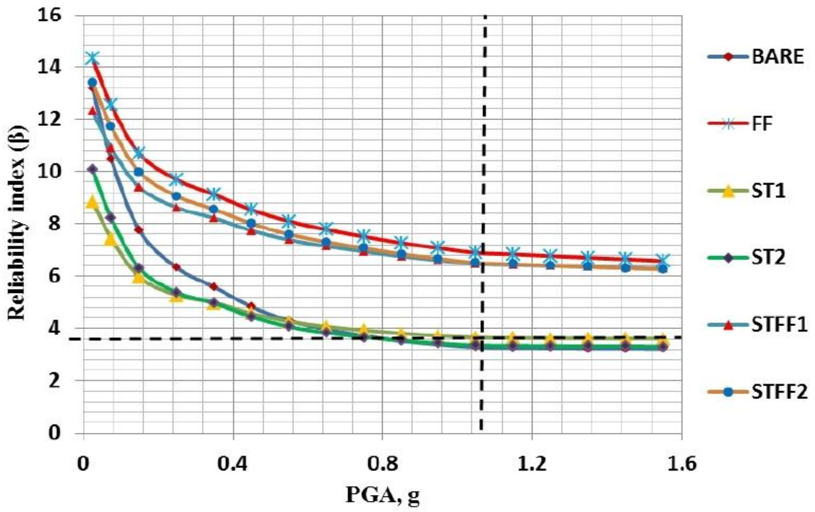

Fig 5 Reliability 8-story, 6-bay, stepped, uneven frames with curves forCP performancelevels

For 6, 8 & 10 storied OGS frames at various per formance levels, likelihood of failure & reliability indices for all frames at PGA of1.05g are determined. They are displayed in Table 1.

Due to greater levels of failure probability, bare frames are discovered to be more prone to failure than other types of frames. In the bare frame analysis, stiffness & strength of infill walls are disregarded. In actuality, the infill walls will provide the structure more strength and stiffness, which improves the building's performance.

As demonstrated in Table 2, full infilled frames (FF) fulfil desired dependability in all performance levels but bare frames (BF) are unable to do so in any of the performance levels indicated by ISO 2394 1998.

The present design process ignores the infill walls during the analysis and design phases.

However, same design methodology might not guarantee required performance for an open ground storey building. OGS1 only just manages to achieve the Target Reliability in all of the performance levels in the current research, while this may not always be the case. This suggests that further study is needed in this area. The fact that the reliability indices for OGS 2.5 are twice as high as the desired dependability suggests that the factor MF may be more cautious. The goal reliability may be used as a foundation for the best design of an OGS construction, especially for design magnificationfactor.

Table 2 Probability of failure and seismic resistance of frames for each limit state for the Manipur

TYPE		6S6B		8S6B		10S6B	
		Pf	B	Pf	B	Pf	β
Bare	CP	0.0033	2.71	6.05 E-4	3.22	5.30 E-4	3.28
FF	CP	2.36 E-11	6.57	2.33 E-11	6.57	2.33 E-11	6.59
OGS 1	CP	7.47 E-11	6.41	9.10 E-13	7.05	7.30 E-13	7.08
OGS 1.5	CP	7.16 E-12	6.76	1.97 E-12	6.95	6.04 E-14	7.40
OGS2	CP	5.47 E-12	6.78	5.01 E-14	7.45	2.85 E-15	7.80
OGS 2.5	CP	3.17 E-11	6.52	5.33 E-20	9.07	5.13 E-25	10.21

CONCLUSION

Reliability indices are derived using the fragility curves created in the previous chapters and seismic hazard of Manipur Region, where building framesare presumed to be located. To assess reliability index, PGA with a 2% likelihood of exceeding 50 yearsis chosen from hazard curve. In order to compare the generated reliability indicesfor each frame (OGS frames & stepped irregular frames), ISO standard suggests comparing them to the goal reliability. The target reliabilities for

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the bare frames are determined to be unachievable. This suggests that the performance of the frames under seismic stresses is greatly improved by the addition of infill walls in the study.

The seismicreliability evaluation of typical verticallyirregular buildings with diverse designs is another major topic of the current work. The hazard curve created by Pallav. (2011) is utilized for the analysis Manipur region & has been displayed by taking into account distinct regions. The observations from the reliability graph are described below.

For 6, 8 & ten storied OGS frames at various performance levels, such as IO, LS, and CP, the likelihood of failure and reliability indices for all frames atPGA of 1.05g are determined.

In contrast to full in filled frames (FF), which are able to achieve the intended dependability at all performance levels, bare frames (BF) are unable to do so. This is according to ISO 2394, 1998.

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