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SEISMIC VULNERABILITY OF REINFORCED CONCRETE HILLSIDE BUILDINGS IN NORTHEAST INDIA

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Abstract

In northeast India, rapid urbanization and limited available land has led to the construction of multi-story, reinforced concrete frames with masonry infill walls on steep hillsides of weak soils. Recent earthquakes in neighboring regions suggest that these buildings may be highly vulnerable to earthquake damage. This paper analyzes the seismic performance of archetypical hillside, reinforced concrete buildings with stepped foundations in Aizawl, Mizoram using incremental dynamic analysis, quantifying collapse risk and identifying potential failure mechanisms. The results show that shear critical columns exacerbate structural vulnerabilities created by stepped hillside configurations. In an earthquake, structural failure likely will begin with axial failure of the half-length base columns at the top of the slope, followed by sequential failures in downslope columns. Collapse is predicted to occur from exceedance of column shear capacity in the stories on stepped foundations. Sensitivity studies of alternative structural and material configurations confirm that current practice of increasing column dimensions at downslope column lines improves lateral strength, relative to uniform column configurations. In addition, utilizing larger transverse reinforcing bars changes column failure mechanisms and increases collapse margin for the expected seismic hazard. The findings demonstrate that improved column shear capacity and above-code detailing may mitigate the seismic vulnerability of Aizawl's hillside reinforced concrete buildings.

Keywords: hillside buildings; stepped foundation; reinforced concrete frames; collapse risk

1. Introduction

Many communities around the world are faced with the challenge of rising populations and limited undeveloped land for new construction. In the city of Aizawl in northeast India, this problem is amplified by the fact that available land for new construction in the city is on steep, mountain sides and ridges. As a result, new residential construction occurs primarily on unstable, weak slopes. Excavations for new sites dug under existing building foundations can increase the risk of landslides, exacerbating an already significant risk due to over-saturation of soils during the monsoon season. The city also has a high seismic risk, due to the subduction of the Eurasian tectonic plate beneath the Indian plate [1] [2]. As the city continues to grow, it is important to assess the seismic vulnerability of the existing and ever-growing building stock, particularly in the light of the recent 2015 Gorkha (Nepal) and 2016 Manipur (India) earthquakes. The threefold goals of this study are to: 1) quantify the collapse risk of typical multi-story reinforced concrete, mixed-use structures in Aizawl; 2) identify the primary mechanisms of structural failure; and 3), investigate how alternative building configurations (in terms of structural and material properties) change the collapse risk and failure mechanisms. To these ends, we develop a computational model of a typical building in Aizawl and analyze the building's vulnerability to ground shaking. Finally, a sensitivity study assesses how variations in structural and material characteristics could improve or worsen collapse resistance.

2. Case study setting

Aizawl is the capital city of the state of Mizoram, which is bordered to the north by the Indian states of Assam and Manipur, to the east by Myanmar, and to the south and the west by Bangladesh. Until 2007, Aizawl was governed by the state, after which a municipal council and then a municipal corporation were established. This city of approximately 300,000 people is thus undergoing a time of significant governmental change and population growth, as city leaders establish a new governance structure and policies. In the case of a major earthquake, the challenges of post-disaster recovery for Aizawl would be immense [1], especially given that the city already has limited economic resources to support basic municipal programs.

The impetus for this study was a seismic risk assessment conducted in 2014 for the city of Aizawl by the non-profit organization GeoHazards International (GHI). GHI has worked since 2012 with Aizawl municipal leaders to design and implement policy and educational strategies for mitigating against landslide and seismic hazards. The observations reported here of structural and geotechnical conditions in Aizawl are the findings of a field study by the first author and informed by the professional experiences in the region of the second and third authors.

3. Background

3.1 Lessons from previous seismic risk assessments and recent earthquakes

In 2014, GeoHazards International (GHI) released a study of the potential impact of a high consequence, rare earthquake on Aizawl. The report details potential economic, structural, and social losses possible from a M7.0 event that generates a peak ground acceleration of 0.35g in the region [1] [3]. The scenario predicts 14,000 buildings would collapse and that earthquake damage would impair roads, impeding emergency vehicle access and isolating many areas of the city. The death toll is predicted to be as high as 18,000 in the dry season, and the magnitude of the losses could increase if the event were to occur in monsoon season, when over-saturated soils make earthquake-induced landslides potentially larger and more deadly.

The outcomes of two recent Southern Asia earthquake events reinforce concerns about Aizawl's seismic risk. On April 25, 2015, a M7.8 earthquake struck Nepal, affecting a mountainous area from Gorkha to Solukhumbu. The earthquake killed around 9,000 people and left hundreds of thousands homeless. Building collapses during this earthquake sequence were dominated by failures of aging unreinforced masonry buildings and weak vernacular concrete frames. Although the earthquakes occurred in the dry season, many people were also killed by landslides [6] [7]. A lack of consensus exists as to the extent of damage to hillside structures with stepped foundations from these events. Post-event reconnaissance by [8] observed major damage to hillside structures at some sites, while others reported that hillside structures did not experience more damage relative to other building types [9]. Review of damage photos [10] shows numerous cases of collapsed hillside concrete frames with well-known vulnerabilities such as weak stories, which may cause collapse before the onset of other hillside failure modes, such as foundation pull-out. More recently, on January 3, 2016, a magnitude 6.7 strike-slip earthquake struck near Imphal, India [11], less than 400 km from Aizawl. Several buildings collapsed from the shaking, while many other structures experienced serious damage and nine people were killed [12]. The occurrence of this earthquake event so close to Aizawl heightens the importance of quantifying the seismic vulnerability of the city's building stock and developing recommendations to mitigate damage from future earthquakes in Mizoram.

3.2 Research on hillside buildings

Although no studies have focused on the vulnerability of the Aizawl building stock, a growing body of literature examines seismic performance of hillside buildings. Paul and Kumar [13] study the stability of slopes underlying hillside buildings. They find that slope safety and stability depends not only on building design, but also on soil strength. From static and dynamic analyses of different building configurations they conclude that heavier building mass should be placed upslope to help stabilize a building under dynamic loading. In a later study, they suggest that when considering building bearing loads, slope stability could be increased by deepening foundation embedment [14]. Birajdar and Nalawade [15] and Singh *et al.* [16] show that shorter upslope columns of hillside buildings typically carry the majority of shear forces and are therefore more likely to experience shear failure. Experimental testing and finite element modeling by Wu *et al.* [17] of the quasi-static performance of multi-story Chinese hillside buildings with stepped foundations also demonstrates that collapse in these structures typically initiates with failures in upslope, ground-story columns. Kharel [18] analyzes hillside, reinforced concrete buildings in Doronka, Egypt and advocates for the use of finite element models to represent soil's complex force-deformation response. Similarly, Farghaly [19] considers how displacement response varies along building height and examines the effect of using beam-on-nonlinear-Winkler foundation (BNWF) models to introduce soil flexibility into soil-foundation-structure interaction models.

4. Hillside buildings in Aizawl

Buildings in Aizawl consist primarily of three types: "Assam-type," "Semi-permanent," and reinforced concrete (referred to throughout South Asia as "reinforced cement concrete" or "RCC"). Assam uses traditional light timber building frames. Semi-permanent buildings are concrete frames with wood floors and either wood or brick infill walls. RCC employs concrete frames and floor slabs, plus brick infill walls, and composes 47% of existing Aizawl buildings, because RCC with masonry infill is low cost and allows for construction of larger buildings on less land [1]. Compared with construction elsewhere in India, many newer buildings in Aizawl have weaker infill walls, often constructed only a single brick wide or with the "brick-on-edge" technique. This practice is sometimes employed to move exterior walls onto cantilevers outside of frame lines to maximize floor space, but chiefly occurs due to a belief that lighter buildings perform better in earthquakes and place less demand on the weak slopes. Since 2007, the Aizawl Municipal Council has mandated ductile detailing for all new buildings to improve seismic performance of RCC frames [1], but there is little to no enforcement of this requirement. Nevertheless, field surveys by GHI suggest that the city has seen relatively high compliance rates with ductile detailing code provisions, due to efforts by local architects and engineers to train masons in ductile detailing and to the propensity of private masons to copy construction practices at government sites. Observations by GHI consultants concluded that most existing RCC buildings, however, are still older, non-ductile frames. Aizawl's steep slopes require most buildings to be constructed with stepped foundations, where individual footings rest on a flat surface, but "step" up the hillside like a staircase at every or every other column line.

Our study analyzes the most common building configuration in Aizawl, RCC frame structures with stepped foundations. The average story height of each of these RCC frames ranges from 10-11 ft. and the majority of multi-use buildings have 5-7 stories. Most new buildings use uniform areas of column and beam longitudinal reinforcement at every floor (#5-#7 bars [Imperial sizes], with metric-sized bars smaller than Imperial size #3 bars in the transverse direction). Concrete grade is typically 2,900 psi (M20) and is commonly hand-mixed. Reinforcement steel has a typical yield strength of 60 ksi (metric grade F415). A common misconception among many construction workers in Aizawl is that engineers often over-design steel reinforcement for beams and columns. Therefore, masons sometimes purposefully place less rebar than specified for the beams and columns in an effort to reduce construction costs. Incremental construction practices are common in Aizawl, where most buildings are constructed in stages, as financing becomes available over time [20] [21]. Incremental construction can lead to rebar corrosion and concrete spalling prior to completion of construction.

Bedrock in the city is predominately sandstone and shale, the top layers of which have been weathered by the tropical climate. Surface to bedrock depth varies, but is often 6-12 ft. (2-4 meters). Given a lack of economic resources for complex geotechnical testing, bearing capacity for an individual site is frequently determined by engineering judgment. Average bearing capacities determined from field tests range between 36 and 51 psi (250-350 kN/m2) [22]. Field reconnaissance suggests there is no standard method for compaction, besides tamping with a heavy rod, nor are the soil bearing capacities used in design well-documented. Foundations are typically spread footings. Footing length and depth (square footing dimensions resting on the ground) range from 3.3 to 6.6 feet (1-2 m). Regardless of building height, the embedment depth of the footing is typically 4-6 ft (1.2-1.8 m).

5. Case study buildings

5.1 Control building design

Based on field observations and literature review of structural and geotechnical modeling practices for this building type, we present a suite of computational models that capture different structural and material characteristics of hypothetical new buildings in Aizawl. The buildings modeled here represent key building characteristics observed in structural plans made available in 2014 by the Aizawl Municipal Council (AMC). The basic building design is an RCC frame, with spread footings on stepped foundations. The study building is six stories, with three bays perpendicular to the hill and three hill-parallel bays, shown in Fig. 1 (a). Each building has a footprint of 39.4 feet by 39.4 feet (12 square meters). Columns are 10.8 feet (3.3 meters) at every story; beams are 13.1 feet (4 meters) in both directions. The effective building weight is approximately 300 kips in all cases, using a dead load of 113 psf and a live load of 20 psf at each floor. The study considers and models only the structural frame, without masonry infill walls (a reasonable assumption given the limited added strength provided by the weak walls typically used as in infill in Aizawl). A key design feature is the stepped footings, which step up and back at each column line, and are all embedded at the same depth, 5.75 ft (1.75 m), relative to the soil surface on the slope.

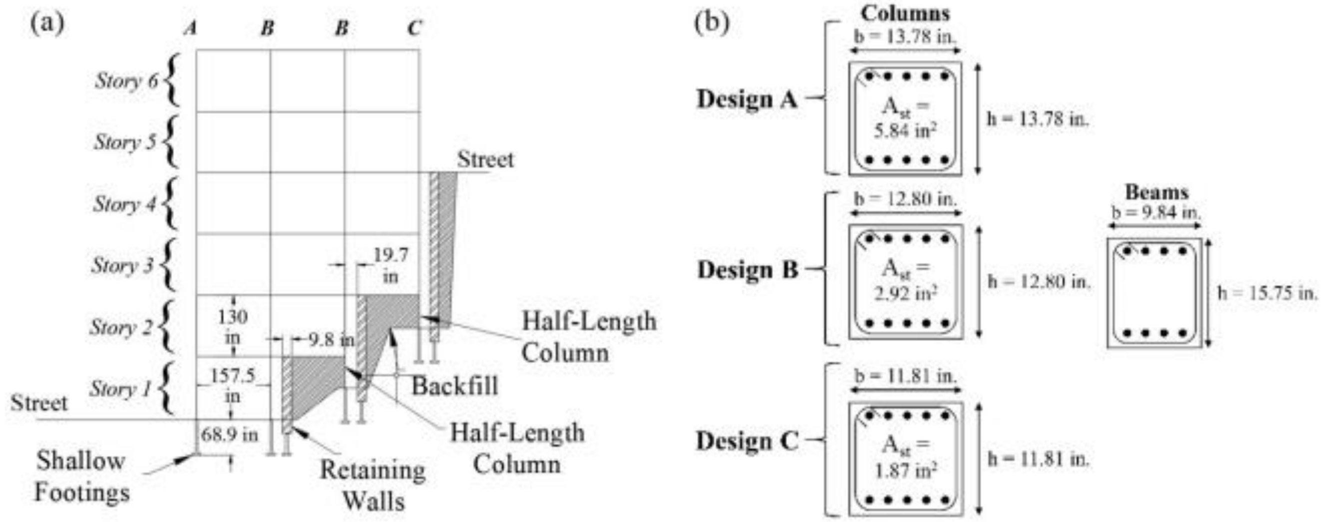

Fig. 1. Control building showing (a) elevation view of building configuration and (b) column and beam dimensions and detailing for case study building (dimensions not to scale).

The stepped foundations are designed such that the ground story columns of the bottom-most bay are the same length at each column line. The first two column lines (C1 and C2) rest on the downslope street level, while the third and fourth (C3 and C4) step up the slope. The base columns on lines C3 and C4 are half the height of the other columns (referred to here as "short" columns). In this study, we define story numbers beginning at the ground story on the downslope side, *i.e.* "story 1" is the ground story at the bottom of the slope. We assume that retaining walls hold back lateral soil forces on the columns. Any lateral or vertical structural support provided by the retaining walls to the frame is considered negligible, although the weight of the backfill and retaining walls is accounted for when modeling soil capacity. Concrete strength and steel reinforcement yield strength follow AMC building plan specifications (2,900 psi and 60 ksi, respectively). Member dimensions on a column line are uniform from ground to top floor, but the as-designed downslope columns are deeper and wider than those upslope. Design detail A is used at column line C1, design detail B is used at C2 and C3, and design detail C is used at C4, as presented in Fig. 1(b). It is assumed that the building was constructed after 2007, thus meeting national Indian design code detailing requirements [23]; the transverse reinforcement uses 0.31 inch diameter bars (#3 Imperial bar size, converted from Metric #8 bar size) in columns and beams, at a center-to-center spacing of 4 in.

5.2 Design sensitivity study

We consider a suite of structural and material property variations, presented in Table 1, to study how seismic collapse risk and specific mechanisms of failure observed in the control model (ID 1) may change with different design choices. The sensitivity variants are: intermediate-sized column dimensions (column design "B"), uniform at all stories and column lines, ID 2; increased concrete strength (f'_c = 5,000 psi), ID 3; increased transverse steel reinforcement (from #3 Imperial bars to #4 bars), ID 4; and intermediate, uniform column dimensions (design B) with flat footings, rather than stepped foundations, ID 5. Model 2, with column design B, is used to demonstrate the effect of increasing the column size and strength at critical upslope columns, but decreasing the column size and strength at the lowest downslope column line (C1). Models 3 and 4 are used to evaluate the potential influence of improvements in construction practices and material quality. Model 5 is used to examine how stepped foundations change the general response and performance of these hillside structures.

Table 1 – Structural and material design variations

Building ID	Column Design	Foundation Type	f'_c (psi)	Stirrup bar size (Imperial)
1 (Control)	A, B, C	Stepped	2,900	#3
2	B	Stepped	2,900	#3
3	A, B, C	Stepped	5,000	#3
4	A, B, C	Stepped	2,900	#4
5	B	Flat	2,900	#3

6. Nonlinear building models

6.1 Structural modeling

Each building variation is modeled in the open-source software platform *OpenSees* using a 2D frame resisting seismic loads along the slope, because we are most interested in quantifying the structural response and vulnerability in the slope-parallel direction. The structural members are modeled with 2D nonlinear fiber beam-column elements attached to zero-length shear and axial springs in series to represent flexural response and possible shear and axial load failures of the columns.. Fig. 2 shows a diagram of the modeled beam-column. Joints are modeled with elastic joint shear panel springs. Rayleigh damping of 5% is assigned to the structure's first and third modes, with damping applied only to the elastic elements of the model. Geometric nonlinearities are accounted for with a P-Δ transformation [29]. The impact of masonry infill walls on the response is not considered.

The fiber sections discretize the longitudinal reinforcement and concrete components into fibers, using a Yassin concrete model that captures linear tension softening [24] and a Giuffré-Menegotto-Pinto reinforcing steel model with isotropic strain hardening [25]. By integrating the stress-strain behavior of each fiber [26], the fiber elements can capture concrete cracking, onset of yielding, and subsequent spread of plasticity along the length and cross-section of the element [27]. Fiber models are an appropriate choice to model flexural response and allow plane sections to remain plane under deformations. However, comparison of flexural and shear column strengths demonstrates that despite following requirements for ductile detailing, the majority of columns shown in the municipal building plans are still shear critical, *i.e.* they are more likely to experience shear failure before flexural failure. Shear failure is followed by axial column failure. In order to represent failure mechanisms of these shear-critical elements we incorporate zero-length springs at the top of each column, with a model developed by Elwood [28]. This model consists of a uniaxial spring that degrades after the detection of shear failure. The limit state model for shear failure detection relates shear demand to drift at shear failure, as a function of the transverse reinforcement and axial load ratios. Elwood [28] also developed an axial spring model to represent column axial failures and loss of column vertical load bearing capacity. We implement both the Elwood shear and axial limit state models here.

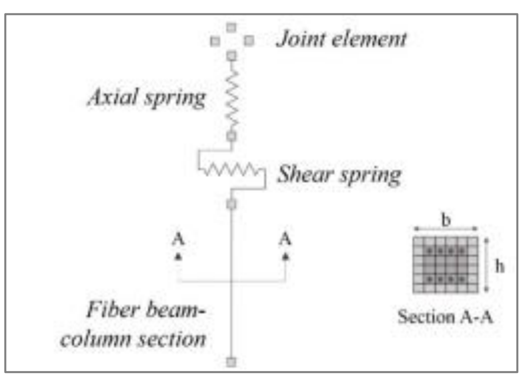

Fig. 2 – Diagram of beam-column elements, joints, and shear/axial springs modeled in OpenSees (not to scale).

6.2 Geotechnical modeling

Previous studies emphasized the importance of modeling soil-structure-interaction in hillside buildings. For stiffer structures, ignoring foundation deformation can lead to unrealistic values for damping and modal frequencies, both of which can mischaracterize seismic performance [30]. Moreover, deformations at the soil-foundation interface can change the overall soil-structure-foundation flexibility, which can increase displacements under dynamic loading and also change the frequency at which the structure responds. Here, the beam-on-nonlinear-Winkler-foundation (BNWF) approach is used to model the soil and soil-structure-interaction. BNWF models utilize independent zero-length soil elements to capture soil-footing interactions, with elastic beam column elements to represent structural footing behavior [31]. We selected the *ShallowFootingGen* command in *OpenSees*, based on a review of existing nonlinear computational BNWF models for shallow footings. The model uses a 2D mesh to connect footing elements to the superstructure beam-column elements [4]. *ShallowFootingGen* constructs elastic beam column elements with 1-D soil springs to simulate vertical load-displacement, horizontal passive load-displacement (against the side of a footing), and horizontal shear-sliding (at a footing base). Vertical springs are distributed along the base of footing models to capture foundation gapping, uplift, and settlement [32].

We calculate the soil and foundation model parameters based on [33]-[37] and using soil bearing capacities from an Aizawl Public Works Department database [22]. We assumed soil model properties based on the limited available data for typical bearing capacity and soil shear strength parameters throughout the city. Our BNWF model provides a simplified method to consider hillside building soil-structure-foundation interactions. The model also assumes constant and linear soil geometric and material characteristics when, in reality, soil properties change nonlinearly under high shear strains, such as those induced by ground shaking [38].

7. Static pushover analysis

First, we assess how variations in structural and material characteristics affect trends of strength, stiffness, and ductility. The fundamental periods and base shear values are calculated from static pushover analyses of the *OpenSees* models. The applied lateral force of the pushover is a triangular distribution.

Table 2 and Fig. 3(a) present the pushover analysis results for each modeling variation. In addition to a comparison of relative lateral strength, the pushover results offer predictions of potential failure modes under dynamic loads. The uniform column model (ID 2) shows that this configuration slightly increases lateral strength, at least under static loads. The results show that, as expected, increasing concrete strength (model ID 3) or the area of transverse reinforcement (ID 4) increases maximum base shear strength. The decreased strength of the flat foundation model (ID 5), consistent with the use of a uniform column configuration, suggests that this foundation design may reduce lateral strength under earthquake loads, although this model does exhibit larger roof drifts (*i.e.* larger deformation/ductility capacity) at the loss of lateral strength than the control. Fig. 3(b) shows the peak interstory drift ratios (IDR) at each column line of the control model. The two upslope short base columns experience much higher IDRs during the pushover than the two downslope full-length base columns. This indicates that under earthquake loads a “zippering” failure mode may occur, whereby the capacity of the base column closest to the upslope street level is exceeded first, followed by sequential downslope failure of the base columns.

Table 2 – Results of pushover analysis

Building ID	**Period T_1 (sec.)[1]**	**Ductility Capacity, μ**	**Max. Base Shear (kips)[2]**	**Roof Drift At Yield (in/in)**
1	2.38	2.30	31.9	0.006
2	2.38	2.15	34.3	0.006
3	1.92	3.91	38.5	0.008
4	2.38	6.46	49.1	0.026
5	3.07	4.00	30.9	0.014

[1] Period from eigenvalue analysis of nonlinear models, considering cracked section properties.

[2] Maximum base shear: maximum base shear per frame line, in units of kips.

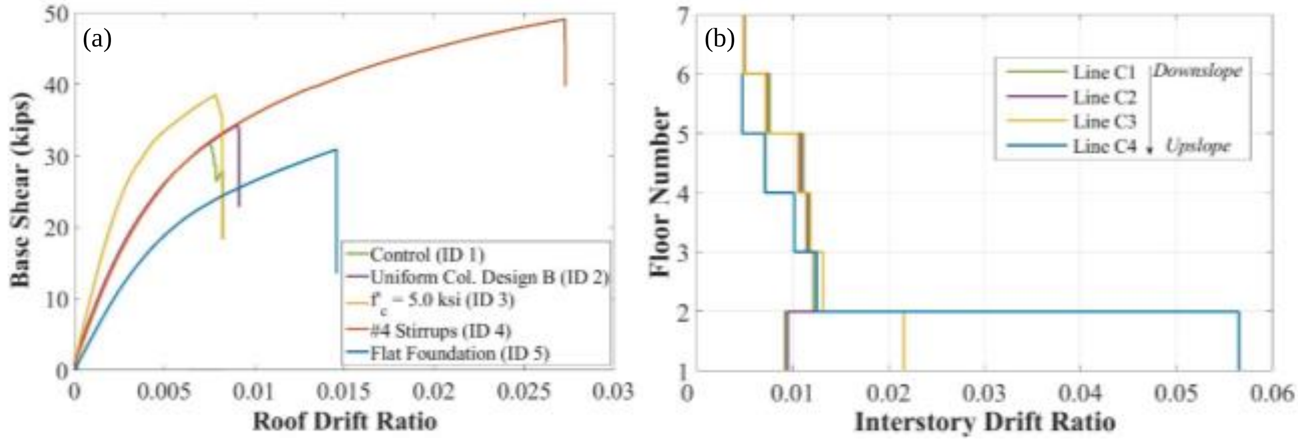

Fig. 3 – (a) Pushover results for all models and (b) interstory drift ratios for column lines in control model (ID1), where "Floor Number" refers to lowest floor at an individual column line (*e.g.* 2nd floor is lowest for line C4).

8. Seismic risk assessment using dynamic analysis

8.1 Ground motion selection

First, a list of 44 earthquake events from Mizoram, northern India, Nepal, Bangladesh, and Western China was identified from databases of historic and recent earthquakes. Searches and processing of time-history data for these events yielded only two usable acceleration time-history records. In some cases, acceleration data were not available, while in others the event occurred too far from a recording station to provide sufficiently large accelerations [39] [40]. To supplement the two regionally-appropriate records selected, information from the 2013 GHI scenario study was used to identify additional ground motion records from historic earthquakes similar to the predicted Aizawl seismic hazard [1]. From the PEER Strong Motion Database, eight additional ground motions were chosen with similar shear wave velocities to Aizawl, between 1,970-2,395 ft/s (600-730 m/s) (from the USGS "Custom V_s30 Mapping"), along with the predicted depth to rupture plane (30 km) and magnitude from the GHI study (M7.0) [41]. Given the challenge in obtaining representative, local ground motion records, we also utilize the set of 30 strong ground motion records listed in [42].

8.2 Collapse fragilities

Dynamic response of the models is assessed with incremental dynamic analysis (IDA). In IDA, response spectra from acceleration time history data are scaled first to a small value of $Sa(T_1)$, then increased at small increments until collapse is observed [5]. Here, we apply three criteria for collapse: 1) if model sidesway results in a peak interstory drift ratio greater than 12%; 2) if shear demand on all columns in a story exceeds the total story shear capacity; and 3) if axial compressive demand of all columns in a story exceeds the total story axial capacity. If no collapse is observed under any of the three criteria, then the analysis is run again at a larger scale factor.

Fig. 4(a) presents the collapse fragility curves for all five models, with ground motion intensities quantified in terms of $Sa(T_1)$. Table 3 summarizes median *Sa* values corresponding to 50% probability of collapse, in terms of both $Sa(T_1)$ and $Sa(T = 1.0s)$. Normalizing the collapse results to the same period, in this case T = 1 second, provides a relative comparison of collapse capacity and avoids variations in response due to ground motion frequency content at different periods. We first discuss the influence of each design variation on the collapse capacities at $Sa(T = 1.0s)$, next we consider how these results compare to the designs of non-ductile buildings in the U.S., and then place these collapse capacities in the context of Aizawl's regional seismic risk.

The uniform column design (ID 2) has the lowest median collapse capacity, in terms of $Sa(T = 1.0s)$, of all study models, suggesting that the upslope variation in column dimensions in the control model is beneficial to lateral load distribution. Using higher strength concrete (ID 3) increases the model's stiffness, decreases its fundamental period and results in a slightly lower collapse capacity at $Sa(T = 1.0s)$ than the control. This finding is consistent with the pushover results for this model (higher base shear strength, lower deformation capacity). The competing strength and deformation capacities of this model, however, may counter-balance each other in terms of their influence on overall collapse capacity. Increasing the size of transverse reinforcing bars (ID 4) significantly improves collapse capacity, perhaps due to greater resistance to shear failures. At $Sa(T=1.0s)$ the flat foundation model (ID 5) has a similar median collapse capacity to the control (ID 1) likely due to: 1) its intermediate column dimensions and 2) the soil-structure interaction modeling assumptions, because its soil bearing capacity is less than for the control (flat foundation case does not consider additional stabilizing pressures from hillside soil loads).

We next compare our results to Liel *et al.* [43], which quantifies collapse risk of U.S. reinforced concrete buildings designed before post-1970s ductile detailing requirements. The results for an 8-story non-ductile perimeter frame (model “8P”) in that study are comparable to the models analyzed here, because it has a similar fundamental period to our control model. Liel’s Model 8P has a median collapse spectral acceleration of $Sa(T_1 = 2.40s)$ = 0.23g, only slightly larger than that of our control model, ID 1, where $Sa(T_1 = 2.38s)$ = 0.18g. Our model with increased transverse steel (ID 4) has a collapse $Sa(T_1 = 2.38s)$ = 0.45g, compared with that of the ductile detailing 8P design variant in Liel *et al.* (0.57g).

Finally, we quantify the seismic risk of the study models relative to a ground motion intensity close to that of the maximum considered earthquake (MCE) for Indian Seismic Zone V [23]. The GHI scenario earthquake and the regional MCE correspond to an approximate spectral response of $Sa(T = 1.0s)$ = 0.40g, estimated from the expected PGA (0.35g) and the median response spectra curve utilized in the IDA. Collapse margin ratio (CMR) is a common metric to assess collapse capacity relative to a specific seismic hazard level, defined as the ratio of median 5% damped spectral acceleration of collapse level ground motions to the 5% damped spectral acceleration of the maximum considered, or scenario, ground motion intensity (MCE) [44]. To correct for the influence on collapse capacity from ground motion frequency content, we compute the “adjusted collapse margin ratio” (ACMR), where the original CMR of each model is multiplied by a spectral shape factor, based on fundamental period and period-based ductility following recommendations from [44]. This adjusted CMR is then compared to recommended values of “acceptable” collapse margins to determine whether probability of collapse at the MCE is less than or equal to 20% given all sources of system uncertainty, as shown in Table 3. 20% is chosen here as the limit because it represents the upper acceptable collapse probability of code-designed US buildings. We compute collapse fragilities based on the expected probability of collapse at MCE, accounting for spectral shape and system uncertainty, and present these fragilities in Fig. 4(b). The new collapse probabilities corresponding to MCE are also reported in Table 3. With the exception of ID 4, none of the case study models meet the threshold level of acceptable ACMR (1.76), thus indicating that, should the MCE occur, their P[Collapse|$Sa(T=1.0s)_{MCE}$] is greater than 20%. ID 4, with increased transverse steel, has an ACMR of 4.96, a far more than “acceptable” collapse margin for U.S. code-compliant buildings, including consideration of all sources of modeling uncertainty.

Table 3 – Median and standard deviation values for collapse capacities (presented at $Sa(T_1)$ and $Sa(T = 1.0s)$ and collapse margin ratio and probability of collapse at maximum considered earthquake shaking intensity, accounting for spectral shape and system uncertainty

Building ID	**Median Collapse $Sa(T_1)$ (g)**	**Collapse $Sa(T_1)$ Std. Dev.**	**Median Collapse $Sa(T = 1.0s)$ (g)**	**Collapse $Sa(T = 1.0s)$ Std. Dev.**	**Adjusted Collapse Margin Ratio[1]**	**P[Collapse | $Sa(T=1.0s)_{MCE}$]**
1	0.18	0.55	0.47	0.91	1.35	65%
2	0.15	0.60	0.32	0.63	1.10	76%
3	0.22	0.43	0.44	0.75	1.54	58%
4	0.45	0.40	1.30	0.53	4.96	6.2%
5	0.12	0.34	0.46	0.69	1.59	56%

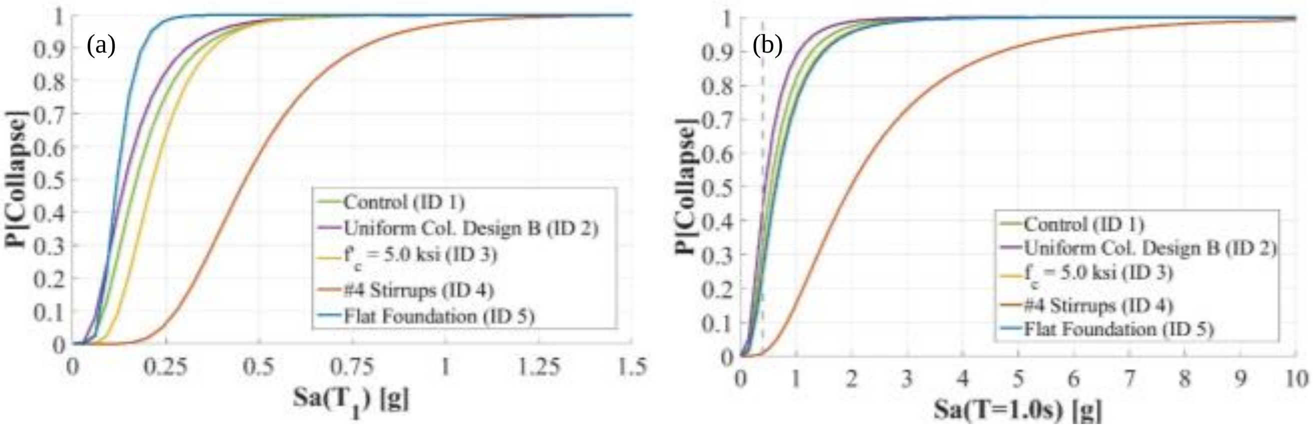

Fig. 4 – Collapse fragility curves for all five hillside models at (a) $Sa(T_1)$ and (b) $Sa(T = 1.0s)$ adjusted for spectral shape and system uncertainty, where dashed line indicates $Sa(T = 1.0s)$ = 0.40g, corresponding to seismic hazard for the maximum considered earthquake in Indian Seismic Zone V.

8.3 Collapse failure mechanisms

One of the main objectives of this study is to identify the mechanisms and sequences of failure for hillside buildings with stepped foundations in Aizawl. Examination of the results shows that the failure sequence varies little between the 40 ground motions. Therefore, we map the failure sequence from the results of the ground motion record that caused the largest number of column failures. Fig. 5(a) shows a graphical visualization of the combined (shear and axial) column failure sequence for the control model. As predicted in the pushover analyses, column failure initiates in an axial mode at the upslope street level base columns, because they are the stiffest and therefore carry large lateral forces. In this damage progression, failure propagates downslope in a sequential "zippering" motion. When failure of one column occurs, the subsequent set of downhill base columns becomes the stiffest and must carry more lateral force. As failure propagates downslope through the base columns, the columns in the 2nd and 3rd stories are required to resist an increasing proportion of the lateral load, before these stories fail entirely, causing the entire building to collapse. Exceedance of the shear capacity in the 3rd story is predicted to be the most common collapse mechanism under seismic loads for hillside RCC structures with stepped foundations in Aizawl.

Fig. 5(b) demonstrates how increasing concrete strength (model ID 3) changes column failure mechanisms, concentrating the majority of failures in the upslope column lines (although the most common collapse mechanisms remains exceedance of shear capacity in the 3rd story). Increasing the transverse reinforcement area (model ID 4) significantly changes the collapse mechanism; for that model, 40% of the ground motions in the IDA result in sidesway-induced collapse, associated with a flexurally-dominated column failure mechanism.

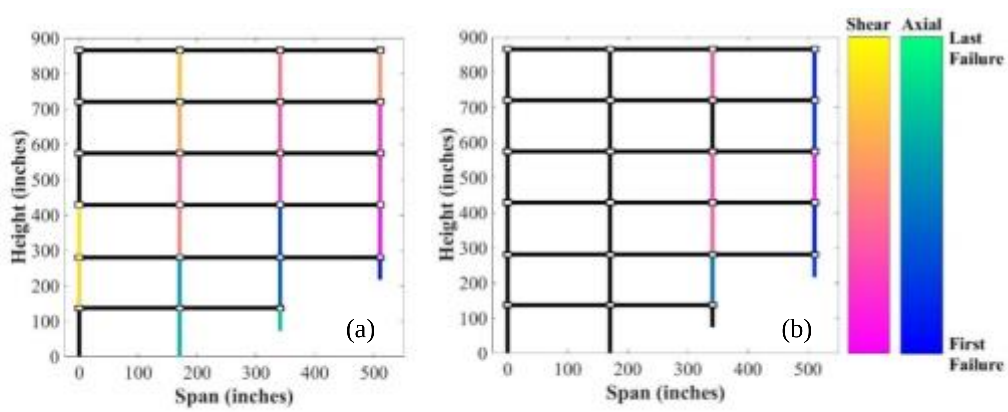

Fig. 5 – Failure sequence for (a) control model (ID 1) showing up to downslope "zippering" associated with base column axial failures and (b) for f'$_c$ = 5.0 ksi model (ID 2) where failures concentrate in upslope column lines.

9. Limitations

The social, economic, and environmental realities of building construction in Aizawl pose significant research challenges to developing models that accurately represent current design and construction practices. We did not consider material degradation from effects of incremental construction, because including these characteristics requires computational models of changes in concrete-steel bond caused by rust or the onset of concrete corrosion, a complicated task due to the scarcity of empirical data. The chosen foundation model is also an over-simplification of the complexity of soil-structure-foundation interactions. To better represent these interactions, future work should utilize more detailed foundation models, ideally validated by field testing of foundation pull-out. Possible other improvements to the foundation model could include: using an equivalent linear procedure to interrogate nonlinear soil shear properties under dynamic loading, accounting for differential settlement, and/or employing a fully nonlinear soil model. We also expect that if the analysis was continued further without convergence issues, all base columns would fail axially (before most upper story columns), constituting a global collapse. Finally, this study assessed only hypothetical new structures; detailed analysis is still needed to quantify the seismic vulnerability and to identify potential retrofit actions for Aizawl's many existing hillside buildings.

10. Discussion and conclusions

This study contributes to a growing body of literature that investigates the seismic performance of hillside buildings with stepped foundations. Despite the above-mentioned limitations, the findings provide insight into the vulnerability of existing reinforced concrete hillside buildings in the northeast India city of Aizawl, by quantifying their collapse risk and identifying specific structural failure mechanisms. The results suggest that typical Aizawl buildings provide insufficient resistance against lateral loads, while as-designed larger column sizes on downslope column lines offer greater lateral strength resistance than uniform column configurations. Our control model has a collapse margin well below the acceptable level for U.S. code-compliant reinforced concrete buildings at maximum considered earthquake intensities, *i.e.* a chance of collapse during MCE shaking greater than 20%. Static and dynamic analyses demonstrate that the short, upslope base columns at street level likely will initiate failure, leading to sequential downslope "zippering" failure of base columns. Structural collapse is predicted to be caused by shear failures in the 2nd and particularly 3rd stories (the stories supported by stepped foundations), as they must resist increasing lateral forces after the base columns failures. A sensitivity study suggests that larger transverse reinforcing increases collapse capacity and changes the collapse mechanism from weak story to sidesway failure.

Recent earthquakes in Nepal and northeast India foreshadow the risk to life and property posed to the city of Aizawl by a future seismic event. Our findings suggest that to reduce vulnerability of new hillside RCC buildings under Aizawl's regional seismic hazard, municipal engineers and government officials should focus on increasing the shear capacity of these buildings and ensuring that such measures are enacted during construction. Future research in this area should investigate effects of specific mitigation strategies, with special attention to toughening critical base columns, providing greater shear reinforcement, and potentially increasing upslope column sizes.

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