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EARTHQUAKE DISASTER SIMULATION FOR ULAANBAATAR, MONGOLIA BASED ON THE FIELD SURVEY AND NUMERICAL MODELING OF MASONRY BUILDINGS

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SUMMARY

Earthquake disaster scenario analysis for Ulaanbaatar City (UB), Mongolia was carried out under the support by UNDP, Mongolia Office and UN Habitat, Fukuoka Office.

To evaluate earthquake risk of the city as quantitatively as possible an extensive approach was adopted. Field surveys for source delineation, ground characterization, building modeling, and building statistics were conducted.

First, seismic risk was evaluated based on the earthquake. Then site amplification was found to not affect strongly seismic motions below 5Hz. From source and site information strong ground motions for two different scenarios, one with large magnitude in a remote area and the other with moderate magnitude in an internediate distance, were generated. Afterwards, building models to evaluate damage ratios due to input motions were build and analysed. In addition, brick wall model was constructed using experimental data to simulate nonlinear behavior of masonry buildings. From such numerical simulations and building statistics we reach the final estimate of devastating damage.

INTRODUCTION

Background

Mongolia has experienced four major earthquakes (M_s>8) and many more moderate earthquakes (M_s 5.3-7.5) in this century. The seismic activity in Mongolia is related to its location between the compressive structures associated with the collision of the Indian-Australian plate with the Eurasian plate on the one hand and the extensional structure associated with the Baykal rift system on the other. The historical records (1903 onward) of the seismicity in Mongolia show a high concentration of seismic activity along the Mongolian-Altay and Gobi-Altay ranges and the north western boarder with Russia and around Mogod east of Hangay mountain. In the process numerous faults have ruptured and new ones are being created thus giving new sources for future earthquakes. While experts believe that the four large earthquakes of

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the century were unusual phenomena with a return period in the order of 5000 years, there are numerous active faults criss-crossing the country that could pose a threat to settlements and infrastructures. The extremely thin population distribution has contributed to a low impact from the seismic activities of the past.

However, because of its shared importance as the political, economic, and population centre of the nation Ulaanbaatar City could face considerable earthquake risk. The City has experienced minor tremors over the past several years due to which public concern regarding earthquake safety is rising.

Ulaanbaatar City supports 849,700 people, which accounts for more than a third of the national population of 2.4 million, produces over 40% of the nation's GDP, and is the junction point of all the country's major roads, railway and air service. As the city is situated on the fluvial sediments of the Tuula River, it is susceptible to damage from earthquakes. The majority of existing pre-80`s housing structures are reasonably well built. However, because of the lack of maintenance and the ongoing alterations in their structures, these buildings could pose a serious threat to public safety in the event of an earthquake.

With the decision to opt for a market economy in 1991, the state apparatuses that were established to carry out and overview constructions and housing have been going through a period of readjustment. Housing construction in general is no longer within the domain of government. This has spurred the construction of detached houses. Many believe that all the tall structures that are being built in Ulaanbaatar suffer from a lack of supervision and the poor use of construction materials. The public is demanding legal protection against "increased earthquake risk" associated with the haphazard construction and the structural alterations of existing buildings. In addition, since the City has to depend on critical facilities like heating plants during winter, there is increasing concern to identify and reinforce or protect such essential facilities as a part of the disaster preparedness.

Scope of the study

The primary aim of this study is to evaluate the earthquake disaster risk of Ulaanbaatar City. This is achieved through the standard earthquake risk evaluation procedure, which consists of four major steps as follows:

1. Evaluation of seismic risk,
2. Evaluation of ground motion,
3. Evaluation of structural damage,
4. Evaluation of social impact.

The first step involves evaluating seismic hazard and developing a scenario of potential earthquakes and their probable frequencies. The second step follows a well-established procedure for ground motion evaluation based on facts and scientifically sound assumptions made in the first step. The third step involves creating so-called vulnerability functions for different types of buildings. The vulnerability functions relate to the degree of damage (e.g., collapse ratios) and the strength indices of ground motions (e.g., peak ground velocity PGV or peak ground acceleration PGA). In the fourth step a complete picture of the earthquake disaster risk is tried to be delineated. It is difficult to predict all the potential impacts before an earthquake occurs. This study highlights major points that are considered weak in terms of earthquake damage potential.

From different types of buildings masonry structures were give a special consideration due to masonry buildings occupy more than 60% of all building stock in UB. For that numerical model for brick wall system was constructed based on experimental data.

SEISMIC RISK EVALUATION

Basically seismicity is relatively low near the vicinity of UB as compared to western Mongolia, where we have many active faults and also many small earthquakes associated with these active faults that generate M8 class earthquakes every 50 to 100 years in that region.

However, there are several faults, some identified only recently while more are being identified, surround the City in the radius of 200 km or less. Together these faults pose a serious threat to UB, although it appears that the recurrence interval could be in the order of a few thousand years. Fig. 1 shows the nearest active fault site Hustayn Nuruu. The fault is running in the Northwest direction toward the UB, which is dominated by normal fault motion. The total vertical offset of the fault seems to be several tens of meters based on the height of the granite hill. According to Dr. Bayasgalan[2], just one event could have produced a vertical offset of 10 meters. The length of the fault could be 30 km or more, and two or more segments could be identified that might be associated with this fault. Since the recurrence rate of this fault could be in the order of several thousands of years, the risk due to this particular fault to UB is not very high. However, it could be one typical example of the scenario earthquakes that one has to consider in the seismic risk assessment of Ulaanbaatar.

Fig.1 Exposure of an normal active fault in the vicinity of Ulaanbaatar.

For the evaluation of peak ground motion the probabilistic approach was employed, using statistical characteristics from an earthquake catalogue for 37 years from 1957 till 1993(over 700 earthquake records). The prediction formulas of peak values or attenuation curves of ground motions was sought. Unfortunately, UB does not have any strong motion stations and thus, no records have been collected to make an attenuation curve. Therefore, it is decided to estimate the probabilistic risk of UB using an attenuation relation suggested by Gotoh et al. [3]. Probabilistic seismic hazard here expresses stochastically the strength of a ground motion (PGA or PGV) that we must expect at a target point within a specified period. If we assume an ordinary probabilistic relationship established in Japan and if we allow ourselves to extrapolate the seismicity data for only 37 years, the PGA value derived from Gotoh et al. [3] formula gives an expected PGA of 220 cm/sec^2 for the return period of 500 years (see Fig. 2).

Based on the information of existence of active faults in the relative vicinity of UB and destructive force of strong ground motion at the source two types of earthquakes, moderate in the relative vicinity and huge earthquake occuring far away from UB, can be considered as a scenario earthquake for the analysis.

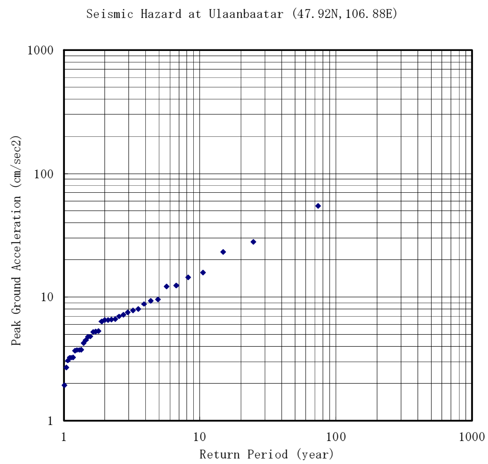


Fig.2 Hazard curve of Ulaanbaatar based on the Gotoh's attenuation formula and the earthquake catalog for 37 years from 1957.

STRONG MOTION SIMULATION

First of all geological structure of UB was evaluated. For this purpose two different methods of microtremor measurements using SMAR6A3P microtremor instruments were combined, that is, a single-station method based on Horizontal-to-Vertical Ratios of Fourier Spectra (H/V ratios), and an array method based on the so-called Rayleigh wave inversion technique used by Kawase et al.[6]. Fig. 3 shows H/V ratio from observed in UB microtremor measurements and theoretical Rayleigh wave (basic mode) H/V ratio. A three-layered structure, namely, 150 m/sec soft layer of 5 m, 250 m/sec intermediate layer of 6 m, and 600 m/sec basement layer has been assumed. This result and the other indirect evidence suggest that the soft soil layers in UB is relatively thin. Therefore, it can be assumed that strong motions that might be observed in UB during future destructive earthquakes would be almost the same throughout the city, except for a very high frequency component, which does not contribute much to the seismic damage of ordinary structures. Fig.4 shows the theoretical soil amplification factor for a vertically incident S-wave. It can be seen that upto 3Hz the amplification of soil can be neglected and at most it cannot be more than 3.5.

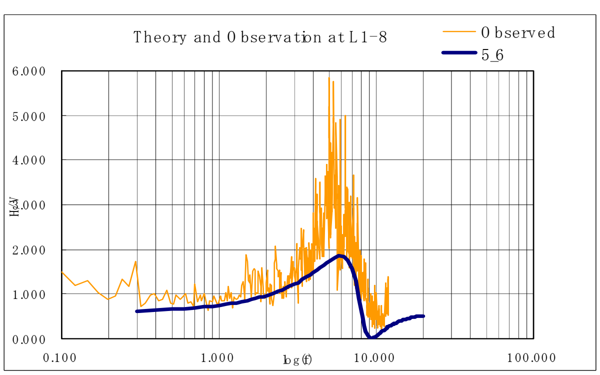


Fig.3 Observed H/V ratio in Ulaanbaatar and theoretical H/V ratio of the fundamental mode of Rayleigh wave for the assumed structure.

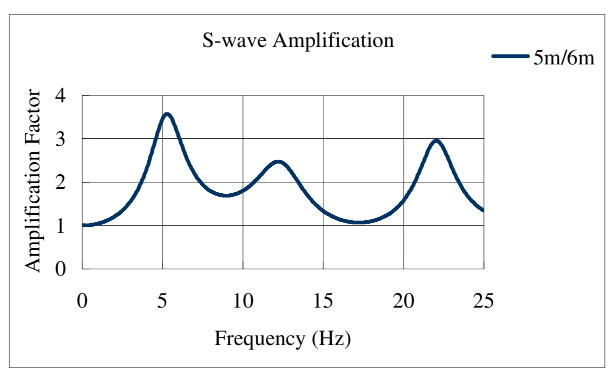


Fig.4 1-D theoretical soil amplification of S-wave calculated for the assumed structure in Ulaanbaatar

Now, assuming scenario earthquake and using the so-called empirical Green's function method by Irikura [5], in which we can account for the source rupture process and the distance effect in a seismologically rational way. From the various studies on seismicity and tectonic setting around UB two types of hypothesized sources were chosen after considering the probability, the magnitude and its impact to UB. These are M8 class big earthquake in the radius of 225 km and M7 class moderate-sized earthquake in the radius of 25 km. Unfortunately, weak motion accelerograms necessary for the empirical Green`s function method were not available. Therefore, although it is not fully desirable, the Green`s function was substituted by the accelerograms observed during the Kagoshima-ken Hokuseibu, Japan earthquake of 1997. Ground motions at many sites in Japan were recorded by a nation-wide seismograph network called K-net [6]. By using these records and the empirical Green`s function hypothetical strong ground motions were obtained for the assumed sources. Synthesized waveforms for the model M7 class fault shown on Fig.5 were obtained and are shown on Fig.6, in which we can see that the mimimum PGA is 130 Gal and the maximum is 380 Gal. Similarly, synthesized waveforms were created for the M8 class fault.

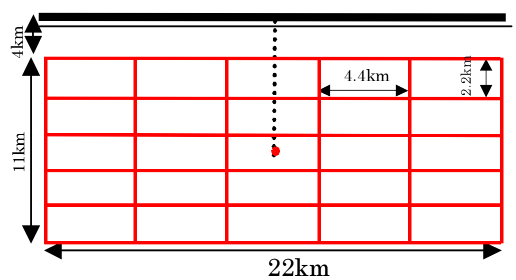


Fig.5 An assumed fault with moderate size in the intermediate distance from Ulaanbaatar

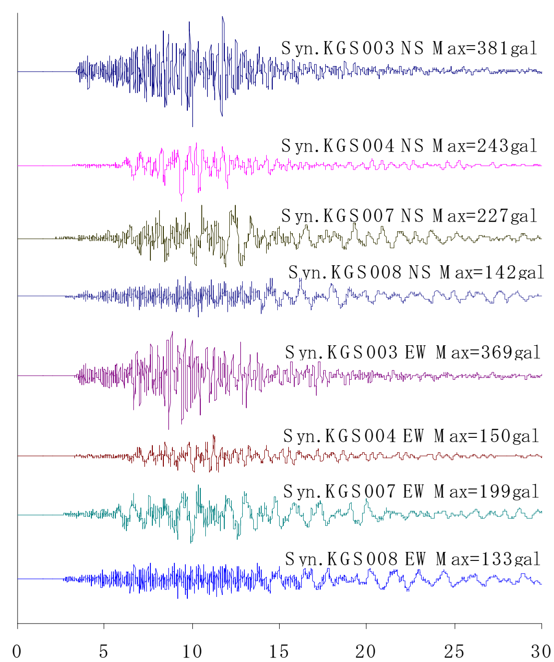


Fig.6 Simulated strong ground motions based on the empirical Green's function method and the source model shown in Fig.5

EVALUATION OF STRUCTURAL DAMAGE

Building survey
Field survey was undertaken to help to understand the real dynamic properties of buildings and to construct the theoretical building models that can simulate their real behaviour during strong earthquakes. These two are necessary for the quantitative damage estimation. Field survey included visiting of construction sites to inspect the standard construction practices and the basic dimensions of the structural elements used for each types of buildings. This was followed by microtremor survey of buildings to reveal their fundamental natural frequencies.

In general there are six major structural types that constitute the stock of buildings in UB. These are masonry structures with thick load bearing brick walls, cast-in-place reinforced concrete (RC) frame structures, pre-cast panel structures that combine several RC-panels of factory-made walls, steel structures, wooden structures and gers (traditional tent like structures). However, the first three building types occupy the main stock of so-called engineered buildings (see Figs.7-9). Masonry buildings in UB, which are usually apartments, appeared sound and seismically safe, but the reckless modification on the first floor to use them as a service space such as shops or restaurants, makes the building weaker for seismic resistance. Pre-cast RC-panel structures are also mostly represented by the apartment buildings. The main weakness of this type of structure seems to lie in the connections between wall panels, which are welded steel bar filled with concrete. This connection could break suddenly once the deformation exceeds a certain threshold. The weackness of cast-in-place moment-resistant frame buildings lies in the length of shear-resistand walls, which are usually very short and poor quality as cast-in-place structural members. Besides these three types of buildings, there are a large number of owner-built, non-engineered buildings such as detached, wooden or masonry buildings, and Gers. These buildings are not included in the risk assessment because of their low vulnerability and for the lack of data on their behaviours under dynamic loading. However, as UB expands it is important to bring such structures within the purview of quality control and technical assistance. The main types of buildings in UB and their relative amount are shown in Fig.10.

Fig.7 A typical old brick building in downtown Ulaanbaatar used as residential appartments

Fig.8 A typical pre-cast panel buildings in the suburb of Ulaanbaatar as residential appartments

Fig.9 A cast-in-place reinforced concrete building under construction.

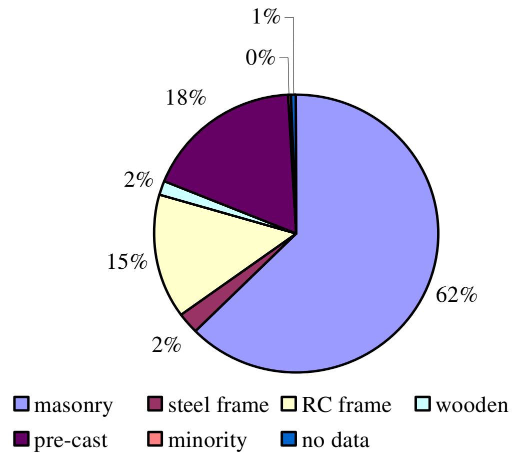


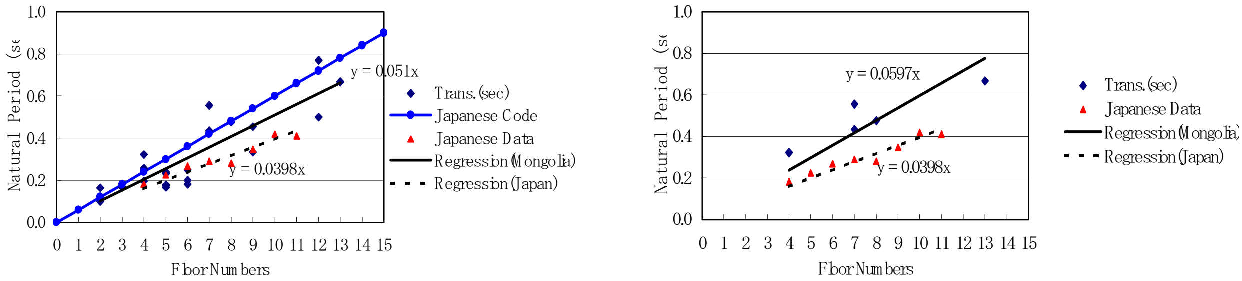
Fig.10 Percentage of different types of buildings in Ulaanbaatar based on the building stock statistics

Evaluation of dynamic charactericstics of the buildings

To reveal the dynamic properties of the buildings, especially the fundamental periods (or frequencies), microtremor survey was carried out. If it is assumed that the deformation limit that corresponds to the ultimate lateral strength of a floor is constant irrespective of building type and age, then the ultimate strength will be proportional to the square of the natural frequency of a building. Thus knowing the natural frequency of buildings is the most important first step toward our final goal of quantitative damage prediction. Since we are mainly interested in the statistical values, not the building-specific values, we need to measure different buildings within the same category. It will also be necessary to construct a relationship with respect to building height (or floor numbers), and so we need to measure buildings with different numbers of stories. In total 28 buildings were surveyed. The purpose of the microtremor measurement was to grasp the dynamic properties of each type of building in UB and to estimate their strength relative to their weight.

As a standard procedure in this kind of analysis the Fourier spectra of the basement and the uppermost floor are calculated for windowed microtremor records and the ratio between them is derived. To determine the peak period of the building the lowest peak period from the spectral ratio is selected and is considered as a fundamental natural period. The same procedure is applied for a short-span (perpendicular to the building's longer axis) component and a long-span (the building's longer axis) component. Thus 56 fundamental natural periods are obtained for 28 buildings that were measured.

Based on all the period data as a function of floor numbers the linear regression line is obtained by fitting it with data following the least square method. Fig.11a shows the regression line together with the natural period data for all 28 surveyed Mongolian buildings and for comparison Japanese reinforced concrete (RC) buildings observed by Imaoka et al. [4]. Also the same graph was plotted only for RC structures in Fig.11b. It was found that the periods for low-rise buildings are almost the same both in Mongolia and in Japan, but those for middle- to high-rise buildings in Mongolia are about 25% longer than those in Japan are. When one translates this into the strength/mass ratio as previously derived, one can deduce that, on average, buildings in Mongolia will have about 64% of the yielding strength of the Japanese buildings. This appears a reasonable estimate. The linear relationship for Mongolian buildings corresponds to $T=0.0510N$ for short-span direction and $T=0.0494N$ for long-span direction. On average Mongolian buildings are stronger than this seismic code relationship suggests.



(a) All buildings in UB **(b) RC buildings**

Fig.11 Predominant periods of the observed buildings in UB and their linear regression line plotted with Japanese data obtained by Imaoka et al. [4] and Japanese code recommendation.

Buildings classified as masonry have only 2, 3, or 4 floors so that we should be cautious when we compare the coefficient with respect to floor numbers. They are T=0.482N for short-span direction and T=0.478N for long-span direction, which is slightly smaller than the coefficients for all the data. Masonry buildings have an average strength when we mix all the data.

Buildings classified to be panel buildings have 5 to 13 floors. Apparently those buildings are stiffer than the average. The regression coefficients for transverse and longitudinal components are T=0.436N and T=0.419N, respectively. This is quite reasonable since panel buildings are constructed by walls on all the external surfaces and walls are quite stiff for lateral forces. However, one should be cautious to take these at face value if we consider the actual strengths of these panel buildings for strong seismic input. As mentioned in the section on the building field survey, a major problem associated with the panel buildings in UB is their heavy dependence on the field welding for connection, which makes their deformability quite limited. It can sustain deformation due to lateral forces to a certain limit. But once the external force exceeds the building deformation limit then all the connections would break apart (the so-called brittle failure).

Buildings classified as RC frame buildings are softer than the average. The regression coefficients for transverse and longitudinal components are T=0.597N and T=0.603N, respectively. Since first RC frame structures are made and then walls are constructed by bricks, the brick walls do not contribute much to increase the lateral strength of buildings. In contrast, in Japan RC walls are caste in place, either as structural elements or just as partitioning walls so that all the walls contribute to improve the performance of the buildings.

In the following study of the non-linear dynamic analysis of building response these average relationships were used to construct building models.

NUMERICAL MODEL OF A BRICK WALL SYSTEM

Two third of the total building stock (62%) of UB city are masonry structures. On the other hand almost all of these structures were designed based on conventional empirical calculation rules. Therefore, a more scientific approach which integrates experimental, numerical and analytical methods for engineering assessment of masonry structures was sought to be important.

Masonry is a composite material consisting of units (brick, stone, block, etc.) and binding material, usually mortar, and there are basically two approaches for the analysis of them: so-called macro- and micro-modeling. In the first approach masonry is considered as homogeneous continuum material embodying combined characteristics of unit and mortar. As for the second approach, besides the fact that unit and mortar are represented as materials with individual mechanical characteristics, unit-mortar interface parameters are introduced as well.

In this study the second micro-modeling approach was attempted. Experiments were conducted on simple specimen to reveal the basic mechanical characteristics of the constituent materials and build a numerical model, as well as on masonry wall specimen, to validate the model. General Purpose Finite Element (FE) code ADINA was used to undertake numerical modeling.

Material model

The masonry used in this was research built from solid clay bricks of size 210x100x60 mm and mortar with C:W:S ratio equal to 1:0.6:2.5. The material mechanical properties of bricks and mortar obtained from conventional tests are summarized in Table 1.

Table 1. Mechanical properties of bricks and mortar

Properties	Mean value	Remarks
Brick		
Modulus of elasticity, Mpa	4,173	Direct compression test
Poisson`s ratio	0.17	Direct compression test
Compressive strength, Mpa	32.50	Direct compression test
Tensile strength (in flexure), Mpa	5.00	Bending test
Ultimate compressive strain	0.0167	Direct compression test
Mortar		
Modulus of elasticity, Mpa	4144	Cylinder test
Poisson`s ratio	0.24	Cylinder test
Compressive strength, Mpa	10.30	Cylinder test
Tensile strength (in flexure), Mpa	1.84	Bending test
Ultimate compressive strain	0.0040	Cylinder test

The deformation characteristics of the brick material and mortar material were obtained from uniaxial compression tests, and the stress-strain relationships used in FE model are shown on Fig.12. Poisson`s ratio was approximately constant up to a stress level corresponding to 78% of the brick strength and 80% of the mortar strength, therefore constant values were employed.

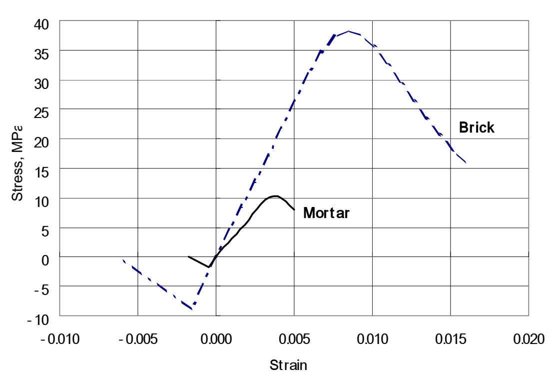


Fig.12 Stress-strain curve used in FE model

Three types of failure mechanisms were applied. These are a three-dimensional tensile failure envelope, triaxial compressive failure envelope, and Mohr-Coulomb failure criteria (see Fig.13). Mohr-Coulomb criteria is defined by two parameters: the cohesion c_u*=53Mpa* and coefficient of friction μ*=0.825*, which were determined from triplet and shear-compression tests.

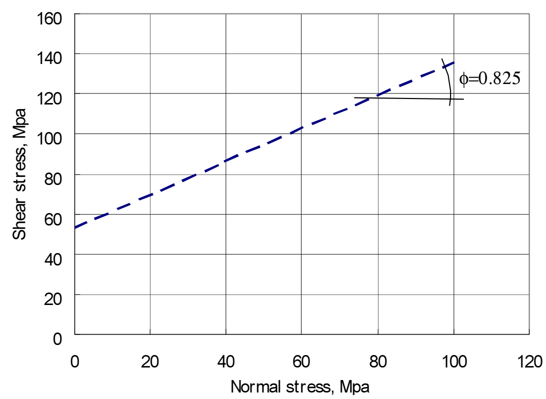


Fig. 13 Mohr-Coulomb criteria for the brick masonry

FE modeling of URM wall

General

Brick and mortar was modeled using 3D-solid 8-node element with different material characteristics. The brick-mortar interface was represented by 3D interface surface element, which is formed of 4-node quadrilateral area segments, and the contact between surfaces are modeled using Coulomb`s friction μ.

URM wall test

In the experimental program by Yoshimura et al.[8] 1750x1190h mm wall, which is approximately 1:2 scale reduced size of actual walls were tested for constant vertical load of 0.84 Mpa uniformly distributed by the loading beam and repeated cycling lateral in-plane load. In the beginning the cycling load was controlling, and then displacement was controlling as the specimen became more flexible (Fig.14)..

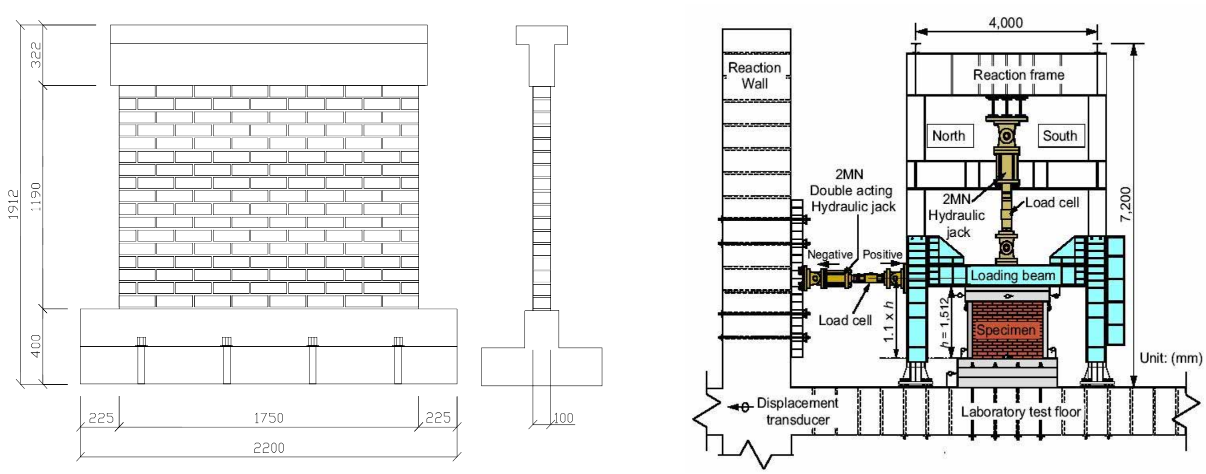


Fig.14 URM wall specimen and test setup [8].

FE model for a wall, consisting of brick elements, mortar elements and contact surfaces is depicted on Fig.15.

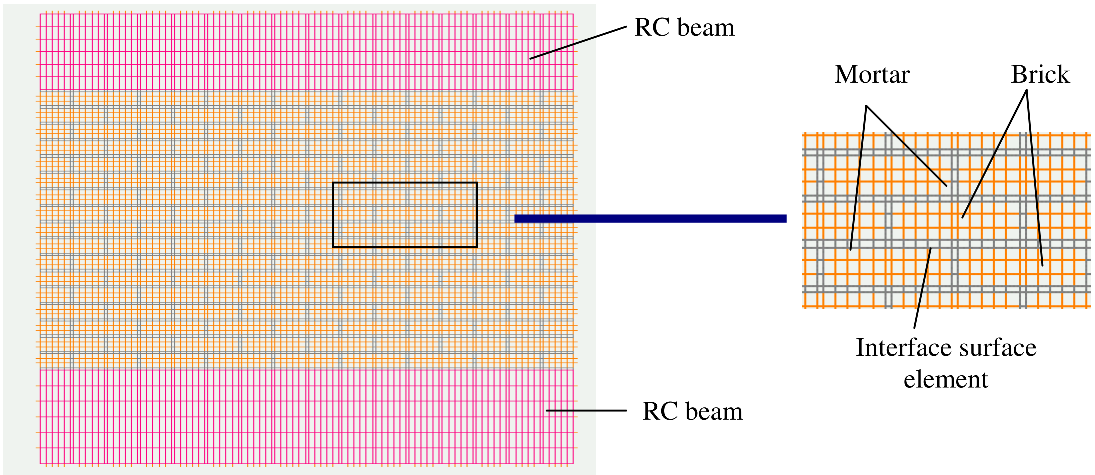


Fig.15 URM wall FE model for analysis

Analysis result

Fig.16 shows the force-deformation relations calculated from the FE analysis of the wall system, and Fig.17 presents crack distribution pattern of the test and crack-stress distribution pattern obtained from the FE analysis.

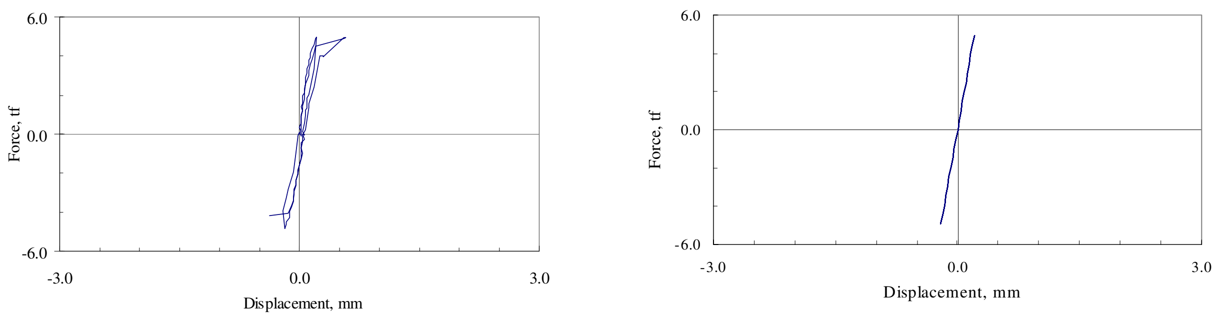


Fig.16 Force-deformation relations for 2 cycles obtained from: a) experiment [8], b) FE analysis.

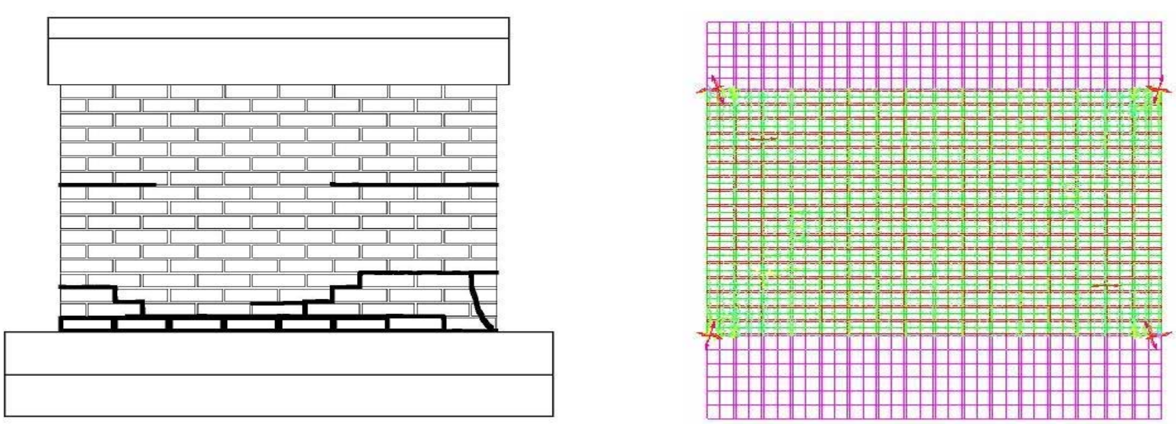


Fig.17 Crack pattern from the test and crack-stress pattern from the FE analysis.

DAMAGE ESTIMATE AND DISASTER SIMULATION OF UB CITY

Now the total impact based on one assumed scenario earthquake among the most dangerous and probable scenario earthquakes is estimated.

First a set of building models with the structural properties of UB were established for damage evaluation. Structural models of a multi-degree-of-freedom system with shear-resisting springs and lumped masses were used. The non-linear behaviour of each shear-resisting spring is represented by a degrading tri-linear model whose characteristics are determined on the basis of the properties derived from the standard design procedure used in a dynamic response analysis in Japan [9]. Unfortunately, the biggest uncertainty in the damage estimate will come from these assumed non-linear characteristics, which is not really proved to be true for buildings in Mongolia. We are referring to the experimental and simulation results of a masonry wall for masonry structures, but we have no information necessary to determine the constitutive relationships for RC and Panel buildings. Therefore, two cases, the best performance model and the worst performance model, have been suggested.

The study uses five representative models of buildings with 3, 6, 8, 10, and 12 floors and assumes that they represent buildings with 1 to 4 stories, 5 to 7 stories, 8 to 9 stories, 10 to 11 stories, and 12 to 14 stories, respectively. Once initial models with stiffnesses and masses derived from existing buildings in UB were created, their stiffnesses were modified keeping an initial shape of the stiffness distribution along the vertical direction so as to fit their calculated natural periods with those of existing buildings. Next it is necessary to assume at what deformation level the building will reach the final yield shear force. Here it is assumed that it would be 1/200 in terms of the inter-story drift angle. There is also a need to assume at what deformation level the building will reach the elastic limit. This is difficult to determine without experimental results (except for masonry), but it was assumed that the elastic limit ranges from 1/5000 to 1/3000.

Non-linear constitutive relationships at the basement derived as such as shown in Fig.18. In this study these models are called Best Performance Models because parameters are primarily based on the field survey of existing buildings in UB and Japanese experimental data. The important characteristic of these models is that they make the estimated damage smaller because the same base shear force will be kept irrespective of the damage level.

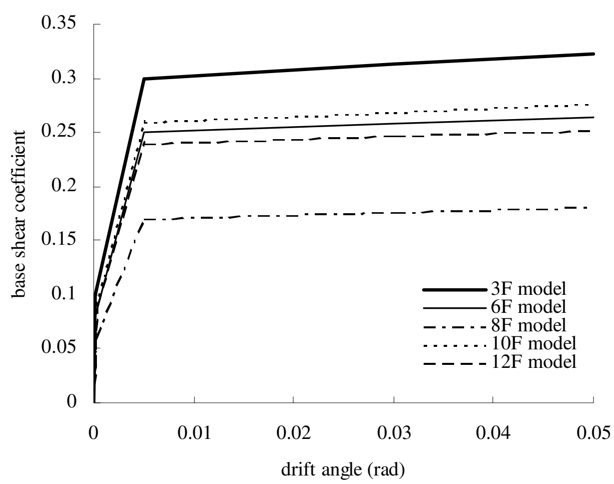


Fig.18 Assumed non-linear skeleton curve based on the design documents and observed periods of buildings

To determine the dynamic non-linear responses of building models and to calculate expected damage ratios a number of assumptions were made: i) the criterion of the maximum drift angle that corresponds to the level of heavy damage, and ii) the distribution of yield strength for the building stock in UB. The former is set to be 1/30, as for the distribution of yield strength a log-normal shape is assumed with a coefficient of variation determined from detailed evaluation of yield strength of many low-rise buildings in Senday, Japan, after Miyagi-ken Oki earthquake of 1978 by Shibata [10]. The probability density distribution of yield strength is shown in Fig.19.

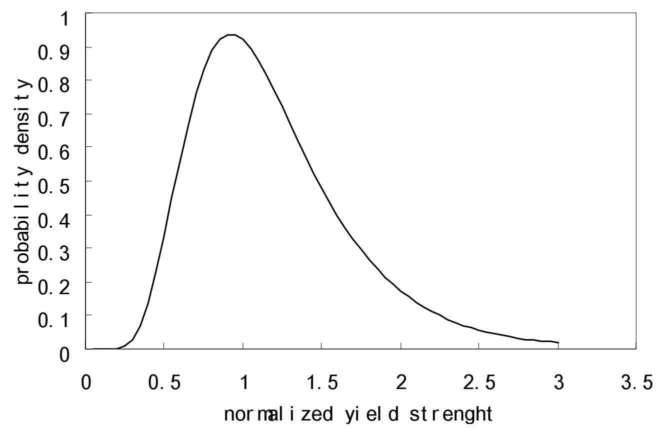


Fig. 19 Assumed distribution of existing probability with respect to the yield strength

The result of dynamic non-linear analysis of building models for the scenario earthquakes in terms of the maximum drift angles are shown on Fig.20, and the distribution of the total number of damaged buildings is shown in Fig.21.

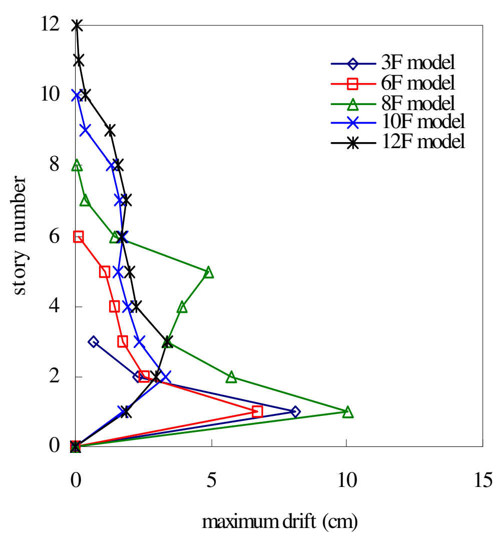


Fig. 20 Responses of assumed buildings for the predicted ground motions due to a moderate-sized scenario earthquake, which are shown in Fig.6.

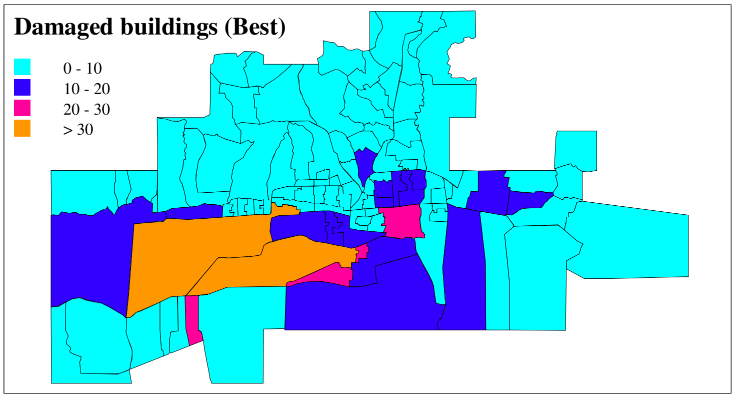


Fig.21 Distribution of numbers of collapsed or heavily damaged buildings based on the nonlinear response analysis for the predicted strong motions.

From the damage estimation the total damage impact of the scenario earthquake can be calculated. The total number of damaged buildings are 700, which is about 25% of the total building stock in UB. Based on this and the casualty probability per one heavily damaged buildings, which is derived from the casualty statistics during the 1995 Hyogo-ken Nanbu earthquake, the total possible human loss was estimated to be 2,400. It must be emphasised here that these numbers of casualties bosed on the damaged building numbers are probably underestimated by quite a large margin since the probability used was the best scenario and thus the least possible loss of life.

CONCLUSION

In this research a first-approximate earthquake disaster risk assessment for Ulaanbaatar city was carried out based on various seismological information, observed building dynamic characteristics and building statistics. The total number of heavily damaged buildings was estimated to be 700 and casualties to be 2,400 even considering the Best Performance case. Recommendations for areas of further studies were outlined as following:

1. Field strong motion observation.
2. Soil structure study.
3. Building strong motion observation.
4. Building deformability characteristics.
5. Methods to strengthen existing buildings.

Moreover, it should be noted here that it is impossible to achieve adequate earthquake disaster preparedness solely by engineering measures. Awareness rising and participation of policy makers, wide-ranging public agencies as well as the private sector and public will ensure an effective disaster risk management.

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