

"Displacement controlled behaviour of soft storey buildings based on the results from full scale testing"

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Abstract:

Traditionally, performance assessment of a structure is based on trading off strength demand with ductility demand. The current guidelines recommend a very low drift capacity for soft storey buildings. Furthermore, the application of these guidelines in low and moderate seismic regions such as Australia will result in most of the soft-storey buildings being deemed unsafe when subjected to earthquake excitations.

Force-displacement data obtained from experimental full-scale studies on a real four-storey soft-storey building in Melbourne has been used as input into nonlinear time-history analyses. A range of near-field and far-field earthquake scenarios of different M-R combinations have been employed for parametric studies. Fragility curves defining probability of failure of structures have been developed. In order to by-pass conventional force based analysis which is time consuming, the new approach based on displacement controlled phenomena has been used for the seismic assessment of structures in low and moderate seismic regions for the seismic assessment.

Keywords: Earthquake, displacement controlled behaviour, non linear time history analyses, and soft storey buildings.

1 INTRODUCTION

1.1 *Background*

In regions of high seismicity, structures are typically designed and detailed to accommodate large displacements without significant lateral strength degradation during an earthquake. Design provisions are based on the concept of conservation of energy, as the kinetic energy developed in the responding structure must be absorbed and dissipated in a controlled fashion. Design guidelines that have been developed in regions of high seismicity (ATC40, FEMA 273) recommend a very low drift capacity for such strength degraded structures. Furthermore, the application of such standards in low and moderate seismic regions such as Australia, results in most strength degraded structures, such as soft storey buildings (Figure 1), being deemed unsafe in an earthquake. In this paper, the recently developed displacement controlled methodology has been used to evaluate factors which can be used as quick assessment of structures for design purposes.

1.2 *Displacement controlled behaviour*

In regions of small and moderate seismicity, the kinetic energy demand generated by a small or medium magnitude earthquake (with M-R combinations consistent with PGV of less than 100 mm/s on rock) will generally subside when the structure has been displaced to a certain limit (Figure 2). More importantly, the diminishing energy phenomenon means that displacement demand is constrained to an upper limit. In other words, the displacement demand on the structure is insensitive to its natural period as the maximum displacement demand level has been reached (Lam et al., 2005). Consequently, according to the displacement controlled phenomena, structures are deemed to be safe, if the displacement capacity of the structure exceeds the imposed displacement demand irrespective of their strength or energy dissipation capacity (Figure 3). Importantly, the ultimate drift capacity of a structure could be defined to be associated with the condition whereby the structure no longer resists gravity load. With this new developed methodology, the displacement capacity of structures can be increased to the limits of incipient collapse (based on the gravitational carrying capacity). Therefore, the displacement controlled behaviour can be particularly beneficial for structures which are conventionally deemed to be non-ductile due to limited energy dissipation capability and significant strength degradation (for example soft-storey buildings).

1.3 *Scope and Objective*

A research project has been undertaken by Swinburne University of Technology in collaboration with The University of Melbourne, Australia which involves experimental field testing of a four-storey soft-storey building in Melbourne. The major aim of this unique project is to study the load deflection behaviour of soft-storey buildings when subjected to lateral loading and to develop a representative seismic performance assessment procedure for soft storey buildings subject to different levels of ground shaking. This paper provides brief introduction to experimental field testing of soft-storey building (refer Section 2). The detailed experimental study has been presented in AEES 2008 (Bhamare et al., 2008). Fragility curves defining probability of failure have been developed using results from non-linear time history analyses (refer to Section 4), which employed synthetic accelerograms of far-field to near-field earthquake combinations (refer to Section 3).

2 BUILDING CONFIGURATION AND TEST SET-UP

The building consisted of four levels above the (open plan) ground storey (refer Figure 4). The upper levels were constructed from precast concrete wall panels whilst the ground floor storey

was built of reinforced concrete columns and beams founded on individual pad footings. Observations from the demolition of adjacent buildings prior to actual testings, indicated that the building to be used for the experimental testing had precast RC columns which had weak connection to the foundation (Bhamare et al., 2008).

Four push-over field tests have been undertaken on a ground floor bay consisting of four columns pre-loaded with kentledge. It was decided for safety reasons to demolish the upper levels of the building to first floor level. A steel frame was constructed at first floor level and positively secured to the slab and beams in order to provide support for the kentledge and to provide anchorage for loading devices (Figure 5). Horizontal loads were applied in both the strong and weak directions via steel tension ties and hydraulic jacks which were secured to a piled tie-back system some distance from the frames. The four columns in a typical bay would need to support around 200 tonnes of dead load plus a proportion of live load from the upper storeys. It was deemed not practical to load the frame with full gravity load and consequently 50 tonnes of kentledge in the form of precast 'jersey barriers' was used. The "slab-on-ground" provided significant restraint to the columns at the ground floor level and consequently two tests were conducted with the ground slab intact. Two additional tests have also been conducted with the ground removed (Figure 5).

2.1 Instrumentation

Various measurement techniques were utilised to obtain the overall load-deflection behaviour of each test specimen as well as curvature of the column and crack width. The loads were measured using load cells. Displacement measurement techniques employed in the field tests included Global positioning system (GPS), total point station (TPS), visual measurement (using a theodolite and ruler), LVDT transducers, laser scanner and photogrammetry. A degree of redundancy was built into the measuring systems to ensure that should one system fail, results could be obtained from other systems.

2.2 Test Procedure and Experimental results

2.2.1 Test Procedure

A hydraulic jacking system with tension ties and a temporary piled tie back anchorage system were used to apply the lateral loads to the frame. The test specimen was laterally loaded by 'force-controlled mechanism' with load increments of 10KN until the ultimate load level has been reached. Further increase in load was applied by a 'displacement-control mechanism' with increments of 25mm to 250mm in each direction.

2.2.2 Experimental Results

A comprehensive set of results have been obtained experimentally. The displacement shown corresponds to the lateral displacement at the slab level and the load is the total lateral force imposed on the structure (Figure 6 & 7). The soft-storey column was found to have significant displacement capacity irrespective of strength degradation. An important outcome of this work is that the columns maintained their gravity load carrying capacity at a lateral displacement of about 260mm (or a drift capacity of about 8%) under quasi-static conditions. Interestingly, the weak connections at the column/foundation and column/beam interfaces allowed the columns to rock about their ends. This resulted in a much enhanced displacement capacity of the soft-storey system compared with rigid-end column connections typified by in-situ construction.

3 GROUND MOTION USED FOR ANALYSES

Accelerograms used for the studies were generated by stochastic simulations of seismological models using program GENQKE (Lam et al., 2000) and SHAKE (Idriss et al., 1992). The simulations comprise various earthquake scenarios of different magnitude and epicentral distance combinations including both far-field and near-field earthquakes. By keeping the magnitude of the earthquake constant epicentral distances were varied from 177 Km to 5 Km. The soil site class C and D were considered having the site natural period of 0.5 and 0.89 sec respectively (as per AS1170.4-2007 classification) (Figure 8). In all 195 M-R combinations on rock, soil class C and soil class D site were incorporated into the parametric studies.

4 SEISMIC ANALYSIS OF SOFT STOREY BUILDING

Extensive parametric studies have been undertaken based on non-linear time history analyses of SDOF of systems. The inputs were taken from the data observed from experimental full scale testing of soft-storey buildings (Section 4.1). This data has been utilised to determine the seismic performance and the drift capacity of a structure in dynamic conditions. The potential seismic performance of the building was assessed by comparing the maximum displacement demand (PDD) with the displacement capacity of the system based on displacement controlled concept. A structure is deemed to be unsafe if the maximum displacement demand from dynamic analyses exceeds its displacement capacity (Section 4.3). Results from the comparison were used to determine the state of failure of the structure. Fragility curves defining probability of failure have been developed (refer to Section 4.2) using results from non-linear time history analyses, which employed synthetic accelerograms of far-field to near-field earthquakes (refer to Section 3). Data obtained from the fragility curves and the rate of failure from each earthquake scenario has been collated. The 5% probability of failure limit has been considered for recommendations for design purposes.

4.1 *Input from full scale testing*

Four test specimens were tested, out of which, two specimens were tested in the strong direction and the remaining two were tested in the weak direction as outlined in Table 4.1. It was observed that the ground slab provided significant restraint to the columns. Thus, two additional tests have been conducted with the slab-on-ground removed.

Table 4.1: Tests description

Test 1	Strong direction with ground slab	Figure 6	Test 3	Weak direction with ground slab	Figure 7
Test 2	Strong direction without ground slab	Figure 6	Test 4	Weak direction without ground slab	Figure 7

The load-deformation data has been used as input for dynamic analyses, where the load-deformation curves were idealised as tri-linear force-deformation model (refer Figure 6 & 7). Rocking behaviour has been generally modeled by the bi-linear (rigid body rocking) or tri-linear (semi rigid rocking) force displacement relationships (Lam et al., 2003). The tri-linear model closely follows the trend of force displacement relationship obtained from the tests. Hence the tri-linear model has been adopted for non linear time history analyses (THA's). This model has two displacement variables: Δ_1 and Δ_2 ; in which the initial stiffness is controlled by Δ_1 and strength degradation controlled by Δ_2 . Interestingly, the observed behaviour of a soft-storey building undergoing large displacement was very similar to that of a semi-rigid rocking object (Lam et al., 2003). The procedure described in Section 4.2 has been followed to construct fragility curves. Results obtained from the fragility curves and discussions have been presented in Section 5.

4.2 Development of fragility curves

Fragility curves are functions that represent the probability of response of a structure to various seismic excitations. Fragility curves are normally presented to predict the probability of damage or failure of a structure/building contents with increasing intensity of applied excitation. The intensity is generally represented by notional peak ground acceleration or peak ground velocity or spectral displacement.

Fragility curves were constructed based on the cumulative probability of exceedance function and the log-normal distribution curve fitting function. Parameters used for constructing the fragility curves were estimated by the likelihood function (L) as expressed in equation 1.

$$L = \prod_{i=1}^N [F(d_i)]^{x_i} [1 - F(d_i)]^{1-x_i} \quad (1)$$

where,

- $F(d_i)$ represents the cumulative distribution function (CDF) estimated at each RSD value, where RSD is spectral displacement.
- x_i is 1 or 0 based on the state of failure (SOF) that corresponds to average RSD_{max} .
- RSD_{max} is the maximum spectral displacement.
- N is the total number of cases considered for the analyses.
- $F(d)$ is written in the analytical form as stated in equation 2.

$$F(d_i) = \Phi \left(\frac{\ln \left(\frac{d_i}{\mu} \right)}{\beta} \right) \quad (2)$$

where,

- “d” represents RSD (Response spectral displacement)
- “ $\Phi[]$ ” represents the log normal distribution function.

The cumulative distribution function (CDF) evaluation is based on a unique value of μ and β which are defined as the median and standard deviation values respectively. These unique values can be obtained by maximising the likelihood function (equation 3).

$$\frac{d \ln L}{d\mu} = \frac{d \ln L}{d\beta} = 0 \quad (3)$$

The procedure described by Shinozuka (2001) was used for obtaining the best-fit curve.

4.3 Interpretation of results

The potential seismic performance of a structure is based on the comparison of the peak displacement demands (PDD) (or maximum spectral displacement (RSD_{max})) of SDOF systems as obtained from non-linear THA's with the maximum displacement capacity of the system. Non-linear THA have been performed on SDOF systems for all combination of earthquakes presented in Figure 8. Maximum displacement demands of SDOF systems observed from non-linear time history analyses were compared with the maximum displacement capacity. Results from dynamic analyses were used to determine the state of failure of the structure. To illustrate, Figure 9 shows the average 5% damped elastic response

spectrum as thick line and the maximum displacement demand collated from non-linear THA as dots. The structure is deemed to be unsafe if the maximum displacement demand from the analyses exceeded the displacement capacity of the structural system. Once the state of failure is known, the combination is then marked either as 1 (or 0) depending on whether the structure is deemed “safe” (or “unsafe”).

Fragility curves which define the probability of failure of the structure were then developed to predict the rate of failure. The computation was performed by the optimisation algorithm using program MATLAB (Figure 10). An example of such a fragility curve is presented in Figure 10, where the dotted points represent the actual percentage of failure whereas the thick line represents the best fitted curve as per the procedure described in Section 4.2.

5 CLOSING REMARK

The force displacement relationship employed in the analyses was based on information obtained from field testing. The field tests and the associated analyses were based on two dimensional response of the building, in which torsional actions are not considered. When the portal frame was tested in the strong direction, cross bracings were used to restrain rotation of the building as well as horizontal movement in the weak direction (such twisting effects could be neglected when the building was loaded in the weak direction).

Various accelerograms were generated and have been employed in nonlinear THA's. More than 14000 nonlinear THA's analyses have been performed. Fragility curves which define the probability of failure of the structure were then developed to predict the rate of failure. The rate of failure of the structure is then used for calculation of the limiting drift demand at 5% probability of failure. This piece of information is imperative to the evaluation of the displacement capacity reduction factor which takes into account dynamic behaviour. Figure 11 - Figure 13 illustrate the variation of percentage failure with respect to the maximum spectral displacement (RSD_{max}) of various ground shaking. RSD_{max} corresponding to 5% failure is considered as the allowable displacement demand for seismic design. Some interesting results have been revealed from this study which is described in the following paragraph.

In summary, the drift capacity of the soft storey column was observed from field test to be about 8% in quasi-static conditions. A more conservative drift limit of about 4% drift is recommended for design purposes (based on 5% rate of failure) in dynamic conditions. It is observed that this drift limit is significantly more generous than the 1.5 % limit recommended by FEMA273 for high seismic conditions. It is noted that this limit is dependent on the aspect ratio of the column, the state of damage to the concrete (little damage was observed in the tests) and behaviour of the foundation.

Secondly, the displacement capacity reduction factor could be derived from these extensive parametric studies. The displacement capacity in dynamic conditions, which accounts for the uncertainties associated with the dynamic loading of the building, can be calculated if the capacity in static conditions and the displacement capacity reduction factors are given (refer to equation 4).

$$DC = SC \times DCRF \quad (4)$$

where,

- DC: Dynamic capacity

- SC: Static capacity
- DCRF: Displacement capacity reduction factor

From the studies it is found that, for quick assessment in design applications, it is not unreasonable to consider RSD_{max} to be (0.55-0.65) times static capacity of the buildings. These recommendations have been suggested which are based on RSD_{max} (corresponding to 5% rate of probability of failure). In other words, the structure could be deemed safe, as long as the value of RSD_{max} is less than or equal to 0.55 times the static capacity of the structure (even for onerous soil conditions).

6 ACKNOWLEDGEMENT

The active support from Delta Demolitions, the Office of Housing, and Fenton Partners in planning and execution of field tests is gratefully acknowledged. The authors are grateful to David Heath, The Department of Civil And Environmental Engineering, The University of Melbourne for his assistance with photogrammetry measurements; and to Dr. Philip Collier and his team, The Department of Geomatics, The University of Melbourne for their assistance with the GPS, total station and laser scanner measurements.

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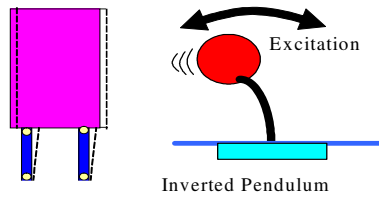


Figure 1 Idealisation of soft storey structures

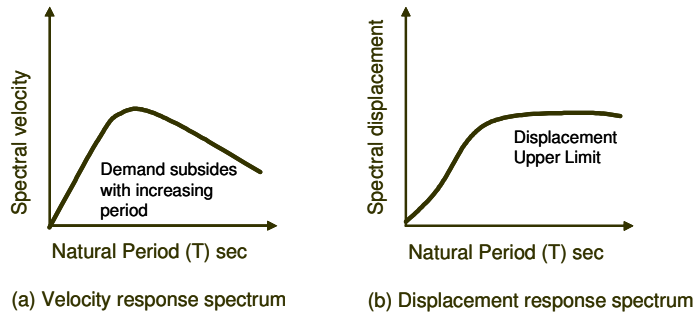


Figure 2 Displacement controlled behaviour

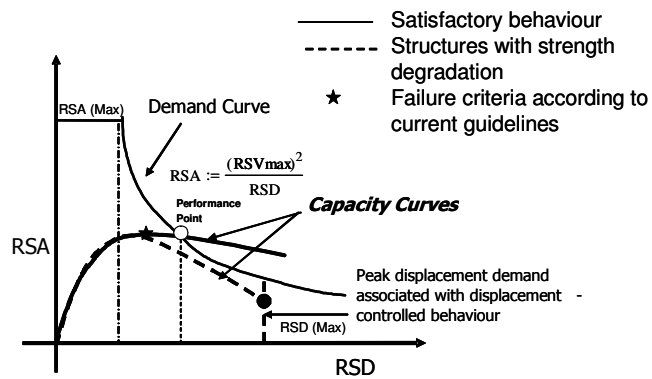


Figure 3 Capacity spectrum method



Figure 4 Photos of soft-storey building

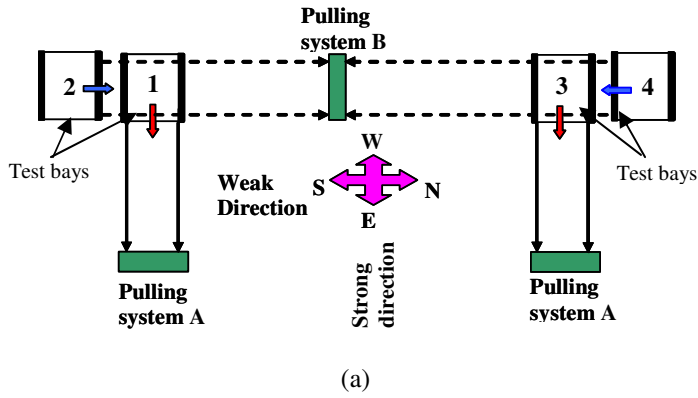


Figure 5 Test overview (a) Test overview; (b) Test specimen

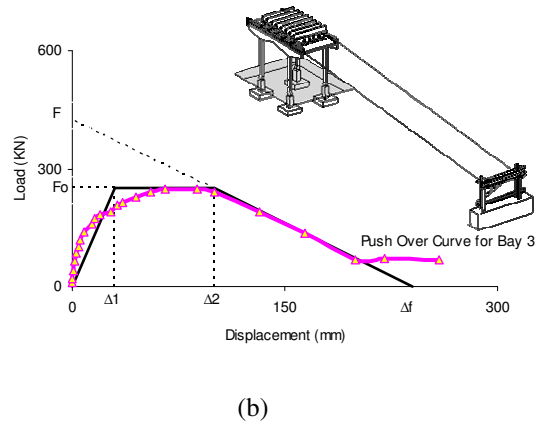
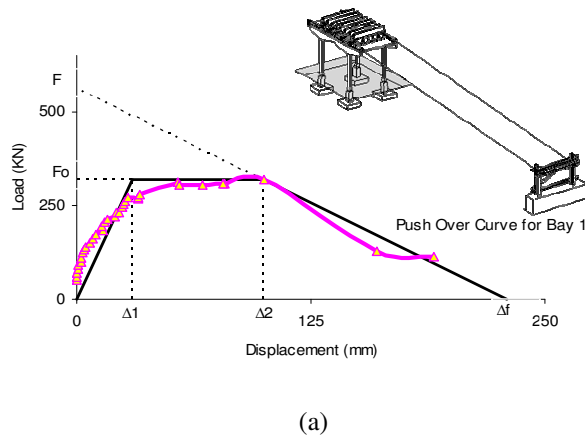


Figure 6 Force-displacement relationships: (a) Force-displacement relationship in the strong direction with ground slab (Test bay 1); (b) Force displacement relationship in the strong direction without ground slab (Test bay 3)

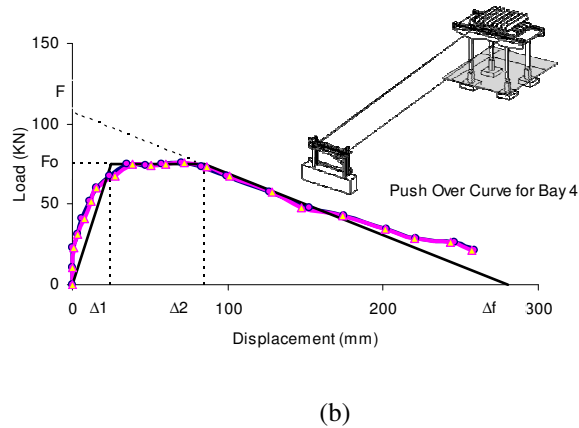
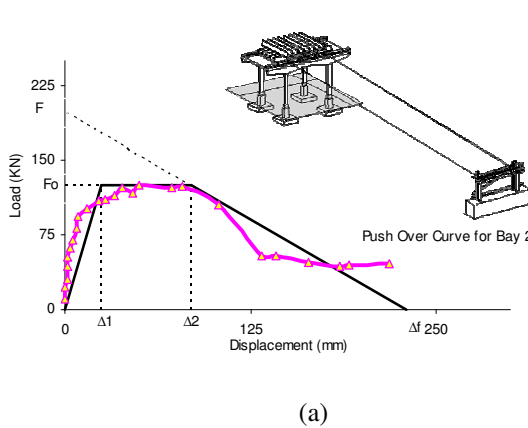


Figure 7 Force displacement relationships: (a) Force-displacement relationship in the weak direction with ground slab (Test bay 2); (b) Force displacement relationship in the weak direction with ground slab (Test bay 4)

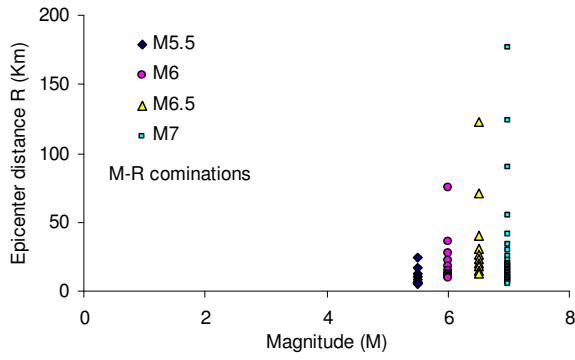


Figure 8 Earthquake scenarios with various magnitude distance (M-R) combinations for site class D soil
 * M: Magnitude of earthquake, R: Epicentral distance in Km

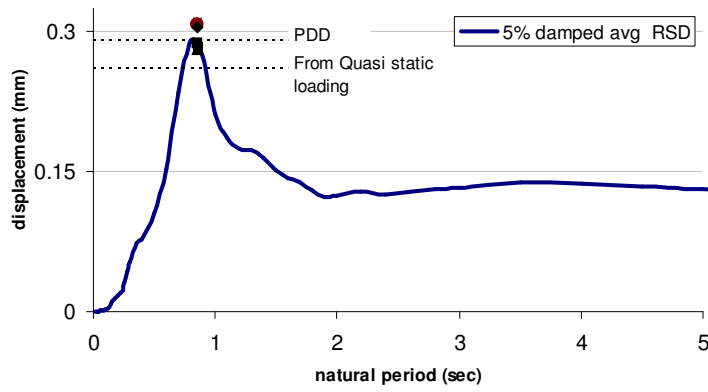


Figure 9 Maximum displacement of inelastic systems (Site class D, M=7 R=15.5KM earthquake)

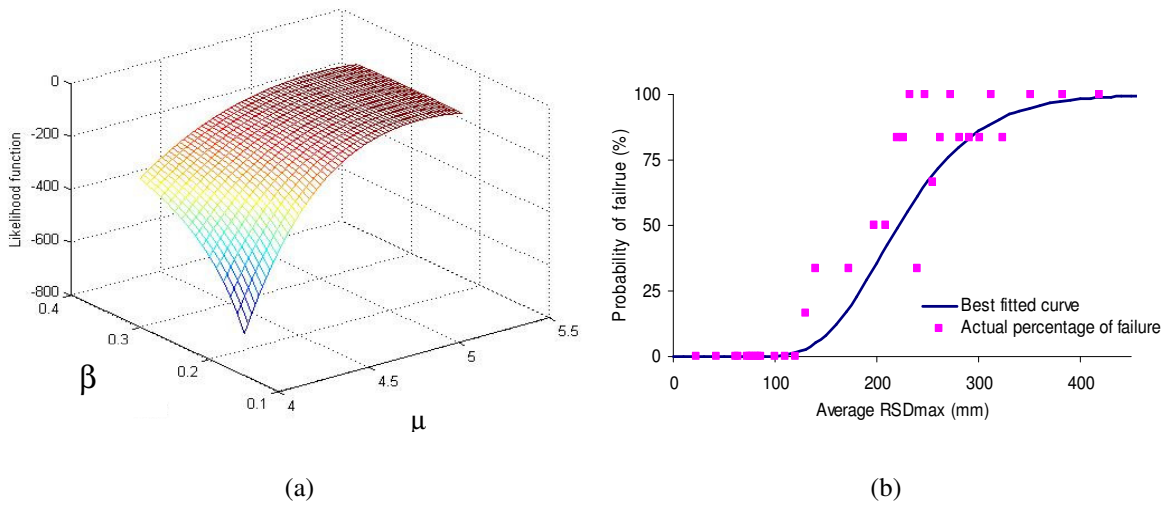


Figure 10 Development of fragility curve: (a) determination of μ and β using optimasation algorithm (b) fragility curve showing best fitted curve

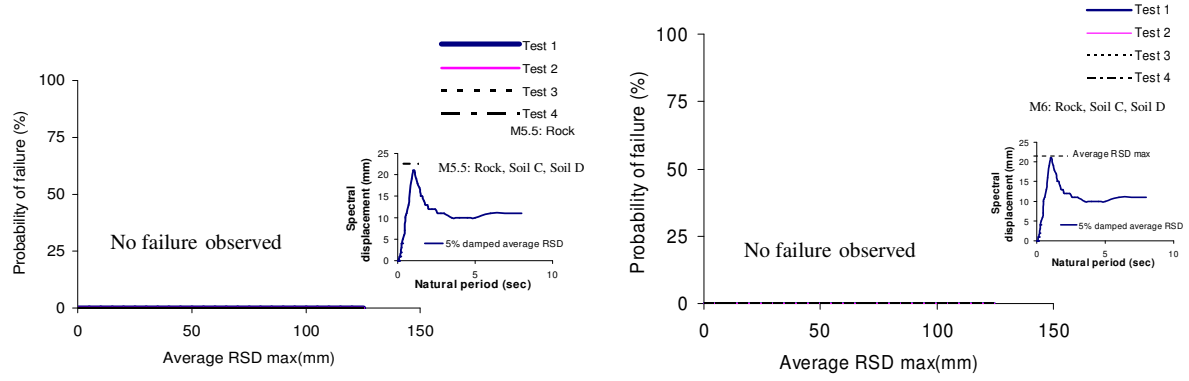


Figure 11 Fragility curves of soft storey building: (a) For M5.5 earthquake on rock site, Soil C, Soil D; (b) for M6 earthquake on rock, Soil C, Soil D

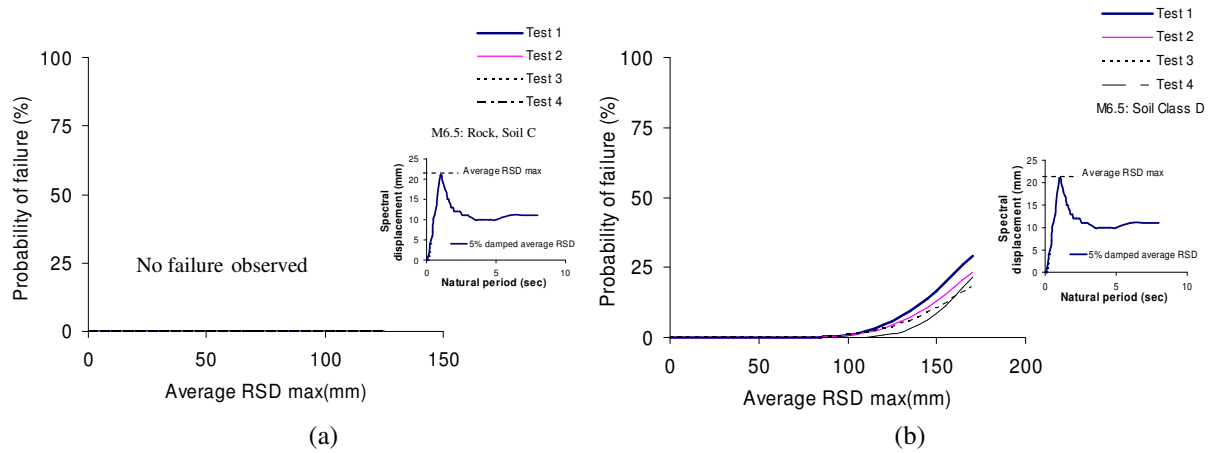


Figure 12 Fragility curves of soft storey building: (a) For M6.5 earthquake on rock, Soil C; (b) For M6.5 earthquake on soil class D

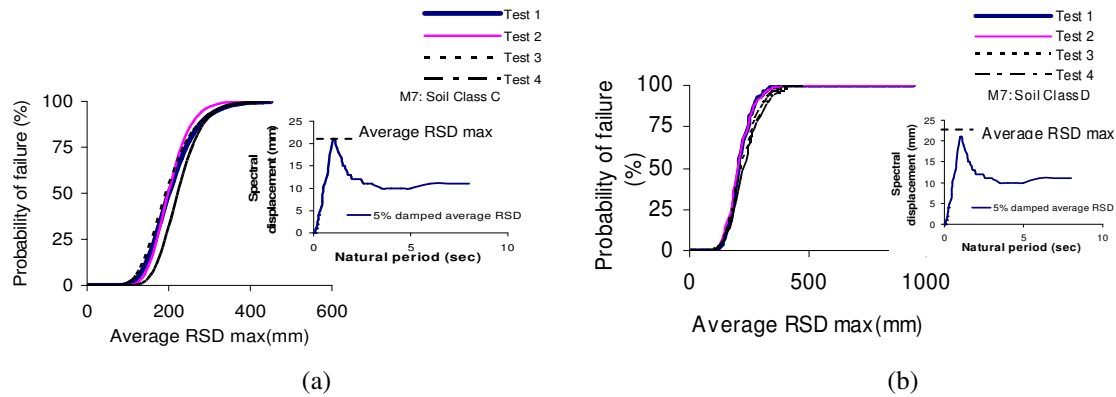


Figure 13 Fragility curves of soft storey building: (a) For M7 earthquake on soil class C; (b) For M7 earthquake on soil class D