MODELLING THE COLLAPSE BEHAVIOUR OF UNREINFORCED MASONRY WALLS DURING THE NEWCASTLE EARTHQUAKE

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ABSTRACT:

The Magnitude 5.5 earthquake which struck Newcastle, New South Wales (NSW) Australia in December 1989 was characterised by widespread damage to unreinforced masonry (URM) walls mostly failing in out-of-plane bending. Damage surveillance identified, amongst other factors, that soft soil sediment sites could have been a major contributor to the disaster through amplified accelerations. The seemingly brittle and low period URM walls therefore appear to be particularly sensitive to the peak ground acceleration. Interestingly however, very high peak ground accelerations (over 1g) generated by a small magnitude event in Eugowra (NSW) in 1994 only caused very minor damage to an URM building situated very close to the epicentre.

This apparent anomaly points to our lack of understanding in the collapse behaviour of URM walls. To implement cost-effective improvement to our URM infrastructure, the physical process leading to their collapse must be fully understood. The conventional quasi-static analysis procedure underlying current code provisions is limited to linear elastic behaviour and would not provide accurate explanations to the highly non-linear behaviour of URM walls undergoing large displacement prior to their collapses.

A simplified version of a new seismic predictive model of URM walls, developed jointly by the University of Adelaide and the University of Melbourne, is introduced in this paper to simulate the collapse behaviour of URM walls during the Newcastle earthquake. It is clearly illustrated in the model that both acceleration and displacement components of an earthquake's excitation are critical to the ultimate behaviour of URM walls and in the absence of either it is unlikely collapse will occur. Ground motion parameters used as input to the model have been derived from a separate study which is described in the companion paper entitled: "Modelling of the earthquake ground motions generated by the Newcastle earthquake".

1. INTRODUCTION

The Magnitude 5.5 earthquake which struck Newcastle, New South Wales (NSW), Australia in December 1989 was characterised by widespread damage to unreinforced masonry (URM). The most common failure mode induced by the earthquake was the transverse bending failure of URM walls. In fact more than 200 parapet and 300 simply supported wall bending failure were reported [1] being often exacerbated by soft and eroded lime mortar joints, poor workmanship and general deterioration. Damage surveillance also identified that 'Soft' soil sites could have been a major contributor to the disaster. This then lead to the common perception that the URM walls were brittle and low period and thus were particularly sensitive to the peak ground acceleration. In contradiction, a small magnitude event in Eugowra (N.S.W.) in 1994, which generated very high peak ground accelerations (over 1g), caused only very minor damage to an URM building situated very close to the epicentre. This apparent anomaly points to our lack of understanding in the collapse behavior of URM walls. To implement cost-effective improvement to our URM infrastructure, the physical process leading to their collapse must be fully understood and applied in rational analysis methodologies.

In recent years displacement based (DB) philosophies have gained in popularity for the seismic design and analysis of ductile structures although have not been thought applicable for seemingly brittle building components such as URM walls. Interestingly, extensive non-linear time history analyses (THA) and shake table testing have shown that face loaded URM walls have a significant capacity to 'rock' without failure [2]. This suggests that DB philosophies might provide a more rational means of analysis than current conventional codified quasi-static procedures. This extended abstract introduces a simplified DB analysis procedure for URM walls, developed jointly by the University of Adelaide and the University of Melbourne having been validated by non-linear THA and shake table testing [3]. The procedure is then used to simulate the collapse behavior of URM walls during the Newcastle earthquake. It is clearly illustrated in the model that both acceleration and displacement components of an earthquake's excitation are critical to the ultimate behavior of URM walls.

2. SIMULATION OF NEWCASTLE GROUND MOTION

Although the historical seismicity of Newcastle is relatively colourful with significant events recorded as far back as 1868 (ML 5.3) there was no operational seismograph in the near vicinity at the time of the 1989 earthquake. Representative ground motions for the Newcastle earthquake must therefore be simulated in order to undertake dynamic rocking analysis of URM walls.

Ground Motion Simulation for Newcastle 'Bedrock' Sites

Eighteen representative accelerograms have been generated for Newcastle 'Bedrock' sites using the computer software 'genqke' for moment magnitude M=5.6 and site distance R=15km. The companion paper 'Modelling of the Earthquake Ground Motions Generated by the Newcastle Earthquake' provides a more in depth description of generic attenuation functions used. For each of the 18 simulated accelerogram specific elastic displacement response spectrum are developed as required for displacement based analysis procedures as described in section (3).

Maximum Spectral Response Simulation for Newcastle 'Soft' Soil Sites

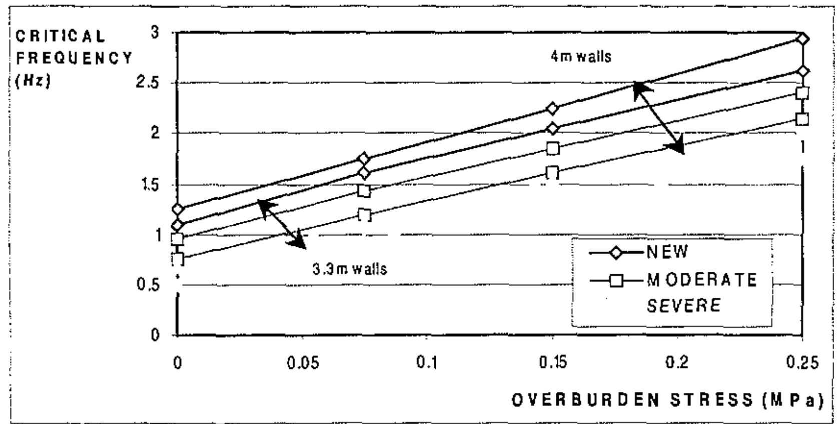
Having determined the 'Bedrock' response spectra the Frame Analogy Soil Amplification (FASA) model is applied to construct 9 Newcastle site specific response spectra that take into account the effects of resonance in the soil, refer Table (1). Further description of the 'FASA' procedure can be found in the companion paper 'Determination of Earthquake Response Spectra in Low & Moderate Seismicity Regions Using New Methodologies'. According to the 'FASA' procedure, the maximum response spectral displacement (RSD) of the soil is proportional to the predicted RSD of the underlying bedrock at the natural frequency of the site. Thus, the 'Bedrock' synthetic accelerograms generated by 'genqke' have been used as input to the 'FASA' procedure.

Table (1) – Newcastle Site Specific 'FASA' Model Results

Newcastle Site Location	Site Natural Frequency (Hz)	Peak RSD (5% Damping)	Peak RSD (3% Damping)
Newcastle Workers Club	1.21	56	67
Hamilton – James & Murray St.	1.28	57	68
Hamilton – Francis Xaviour	1.31	77	92
Newcastle RSL	2.31	43	52
Franklins (old store building)	2.13	47	56
Newcastle Technical College	1.45	56	66
Taxation Office	2.94	30	36
Tudor Inn Motel	2.74	34	40
Ambassador Hotel	1.12	83	98

3. DISPLACEMENT BASED METHODOLOGY

The displacement-based (DB) methodology is based on the comparison of displacement capacity and predicted displacement demand. To simplify complicated non-linear behaviour both displacement demand and capacity are defined using a 'substitute structure'. According to the 'substitute structure' procedure the response displacement of a SDOF system (as indicated by the RSD) can be used to represent that of a free standing parapet (cantilever) at two-thirds of the total wall height. Thus, the top of the wall displacement can be taken as 1.5xRSD. A similar scaling relationship can be used to predict the displacement of the mid-height of a wall spanning vertically between top and bottom wall supports. It can be shown that both types of URM wall described above will be safe from instability and failure provided that the wall thickness, representing displacement capacity (typically 110mm for a single leaf URM wall), exceeds 1.5xRSD representing the maximum displacement demand. If however at a given frequency the peak displacement demand exceeds the displacement capacity a more detailed analyses considering wall specific critical rocking frequency are required to determine the walls seismic behaviour and ultimate stability performance. For a rocking URM wall the critical rocking frequency can be related to the walls resonant rocking frequency at which maximum displacement amplification occurs. This in turn is related to a wall's height, overburden stress and mid-height joint degradation. Figure (1) presents critical rocking frequency for a simply supported URM wall considering the above defining parameters as determined by THA parametric studies. Importantly the critical rocking frequency increases with overburden stress, decreases with mid-height joint degradation



and wall height and is independent of wall thickness.

Figure (1) - Critical Rocking Frequency for Simply Supported URM Walls

4. ANALYSIS FOR SIMULATED NEWCASTLE GROUND MOTION

Analysed Walls

URM walls selected for analysis are typical of those found to behave satisfactorily on 'Bedrock' sites in Newcastle but observed to fail at certain 'Soft' soil locations. Walls included 3.3m and 4m tall, 110mm thick and at overburden stress levels ranging from 0MPa to 0.25MPa. Three levels of mid-height joint degradation were also considered ranging from new to severe. The reasoning behind the selection of a single leaf 110mm thick wall for analysis was that the majority of reported failures were attributed to cavity wall tie failure. This resulted in each of the two leaves effectively behaving independently generally resulting in the non-loadbearing leaf failing.

'Bedrock' Site Analysis

Intensive non-linear THA were completed for the 18 accelerograms simulated for the Newcastle 'Bedrock' site ground motion for each of the wall configurations. In agreement with observed behaviour the displacement response indicated that none of the walls would have become unstable. The dynamic analysis was then repeated using the simplified DB procedure, refer Figure (2). Here the horizontal line represents the displacement capacity of the 'substitute structure' (73mm). Below this are the 18 simulated displacement response spectrum representing the range of possible displacement demand. As for all critical wall frequency the peak RSD does not exceed the displacement capacity. The DB analysis therefore again confirms that no wall configuration would become unstable as per the observed behaviour.

'Soft' Soil Site Analysis

With reference to Table (1) the peak RSD representing the maximum displacement demand of the 'substitute structure' and the corresponding site frequency, at which the peak RSD occurs, are presented for 9 'Soft' soil sites in the Newcastle area at both 5% and 3% damping. Typically peak RSD vary from 30-100mm depending on the site natural frequency that in turn depends on the depth of the soil. Thus, for some sites the peak RSD exceeds the displacement capacity (73mm) indicating instability although for other sites the capacity exceeds demand indicating stability. Therefore, as was observed in Newcastle, certain wall configurations would be expected to fail on some 'Soft' soil sites and not others. For 'Soft' soil sites with RSD greater than 73mm the corresponding site frequency ranges from 1.12Hz to 1.31Hz. Again referring to Figure (1) we can see that walls with critical frequencies in this range and as such which would respond to the peak RSD are non-loadbearing (OMPa overburden stress) walls. Therefore it can be concluded that the analysed single leaf 110mm non-loadbearing walls would have been expected to fail during the Newcastle earthquake at certain 'Soft' soil sites but not others. This is again in agreement with observed failures.

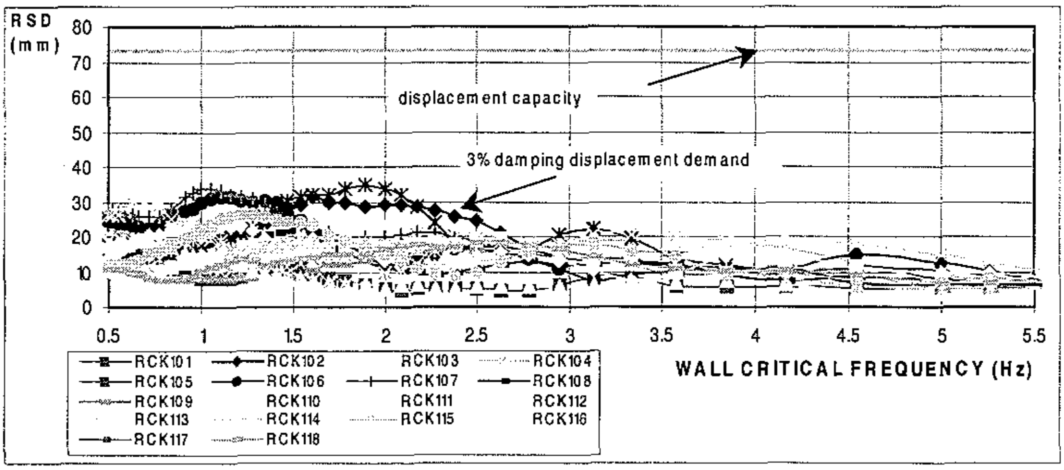


Figure (2) 'Bedrock' Site DB Analysis

5. SUMMARY

A simplified DB analysis procedure has been presented for face loaded URM walls validated by extensive THA and shaking table tests. Simulated Newcastle earthquake ground motions for both 'Bedrock' and 'Soft' soil sites were then used to analyse typical URM walls with good correlation found between analytical and observed failures. This study reinforces the effectiveness of the DB methodologies for the analysis of face loaded URM walls. Further, it dispels the common misunderstanding that stability of URM walls is solely dependant on the instantaneous acceleration or inertia force which is assumed in traditional codified quasi-static analysis procedures but rather on a combination of both ground acceleration and displacement.

6. REFERENCES

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