Alternative Approach to Analysing Infrastructure Using Limited Acceleration Time History Analysis

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Abstract—**Various methods are used to analyse infrastructure subjected to seismic loading. These range from push over analysis to acceleration time history analysis. The acceleration time history analysis is widely regarded as a superior method for analysing infrastructure in seismic prone regions. However, the disadvantage is that this method can be computationally expensive depending on the size of the structure as well as the number of and length of the acceleration time histories used. The traditional approach also chooses the largest value of a parameter, i.e. shear force or bending moments, as the maximum value, which could lead to significant inaccuracies. The proposed method uses two acceleration time histories based on a minimum and maximum intensity earthquake which is obtained from the displacement profiles for a particular peak ground acceleration, which uses less acceleration time histories compared to the traditional approaches. A "picking" algorithm is also used to determine the maximum parameter magnitude thereby eliminating the possibility of choosing outlier values. This leads to the method providing a minimum and maximum parameter magnitude, leading to a parameter force band. Once the design capacity of a section is known and superimposed with the force band, it allows the design engineer to immediately visualize the robustness of a section.**

Index Terms—**PGA, acceleration time history analysis, force band, maximum force**

I. INTRODUCTION

Certain regions in the Western Cape Province of South Africa are susceptible to moderate intensity earthquakes up to 0.15g [1]. Recent research indicate that these regions are susceptible to earthquake magnitudes up to 0.23g [2]. A significant percentage of the infrastructure located in these areas were constructed prior to the first loading code, SABS 0160 of 1989, which propose guidelines for seismicity design [3]. This, therefore means that these infrastructure were not designed for seismic effects. A new seismic loading code, SANS 10160-4, dedicated to seismicity based on Eurocode, EN 1998-1:2004, replaced SABS 1060 in 2011 [4]. With the implementation of SANS 10160-4 and the discrepancies between the maximum Peak Ground Accelerations (PGA), concerns were raised whether infrastructure located in these regions are robust to resist the additional forces generated through moderate intensity earthquakes. A series of investigations were therefore conducted to determine the robustness of various types of infrastructure [3], [5]-[9].

These studies were conducted using the acceleration time history analysis. None of the researchers used the same acceleration time histories in their analysis. Thus, the validity of the results could possibly be questioned based upon the type and magnitude of acceleration time histories used. Therefore, a more robust way of evaluating infrastructure must be used when using the acceleration time history response, with specific reference to choosing the acceleration time histories.

A study by Solms under the guidance of Haas, was conducted to determine the robustness of an important bridge, namely the Stellenberg Interchange, which crosses a national road leading into Cape Town in South Africa [8]. The bridge is curved in plan with a span of 418m, has a radius of 245m and is supported on 13 columns including the support abutments, which is shown in Fig. 1. For a more detailed description of the bridge the reader is referred to Solms [8].

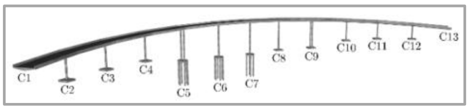

Figure 1. A representation of the stellenberg interchange.

An exploratory investigation of the Stellenberg Interchange was therefore conducted due to;

- The uncertainty of the maximum possible earthquake magnitude.
- The soil conditions at the site.
- The soil conditions from the possible epicentre to the interchange.

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- The bridge not conforming to modern day best practices for bridges located in seismic prone areas.
- The bridge not designed to resist earthquake loading based upon the design code used at the time.

Various options were considered for the analysis of the bridge, ranging from the push over method to the non-linear acceleration time history method. Since no previous analysis work was performed on the bridge and the complex nature of the reinforcement, the push over analysis was not feasible. Also, it was important to determine the displacement profile of the entire bridge, the shear forces and the bending moments at certain locations in the columns and bridge. It was also important to determine the mode shapes and corresponding natural frequencies of the entire bridge. Thus, the only option to use was the Finite Element (FE) method.

It was therefore decided to perform the analysis using an acceleration time history response applied to a Finite Element model developed in ABAQUS to determine its response during a typical earthquake. This approach would allow the user to obtain the unknown parameters if beam elements are used. The stresses and strains at any location in a section can only be obtained if solid elements are used which would make the analysis computationally expensive and inefficient.

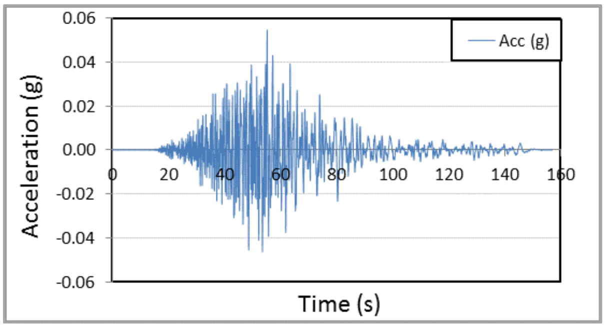

Figure 2. “Minimum” intensity earthquake for 0.05g. Station P1524 North. [9].

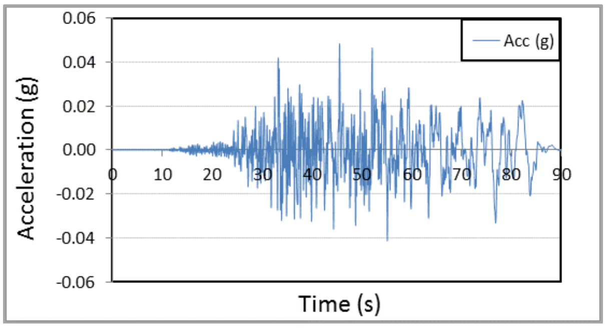

Figure 3. “Maximum” intensity earthquake for 0.05g. Station P1155 North. [9].

The problem which we were faced was to obtain realistic PGA’s with magnitudes ranging from 0.05g to 0.25g in increments of 0.05g. Several websites are available where these time histories can be downloaded. It was decided to use the Chi-Chi earthquake which occurred in Taiwan in 1999 since detailed acceleration, velocity and displacement time history records are available. After careful review and plotting the displacement profiles of various stations with similar magnitude accelerations it was observed that the displacement profiles varied significantly. Fig. 2 and Fig. 3 shows the acceleration time histories for a PGA of approximately 0.05g. Fig. 4 and Fig. 5, shows the respective displacement histories for these PGA histories. It is clear from Fig. 4 and Fig. 5, that although the earthquake magnitudes are similar, it yields significantly different displacement profiles. This could therefore lead to confusion as to which acceleration time histories should be selected to conduct the FE analysis to conform to EN 1998-1:2004 clause 3.2.3.1.2 4(a) and other codified requirements [10], [11]. EN 1998-1:2004 requires a minimum of 3 acceleration time histories while NIST requires a minimum of 30 be used in a non-linear analysis [10], [11].

The displacement profile from Fig. 4 yields an absolute displacement of 116.2mm, while Fig. 5 yields an absolute displacement of 286.0mm. This results in a difference of 246%. The displacement response of Fig. 4 is very jaggered during the initial phase. However, the displacement response in Fig. 5, although it produces larger displacements, is much smoother.

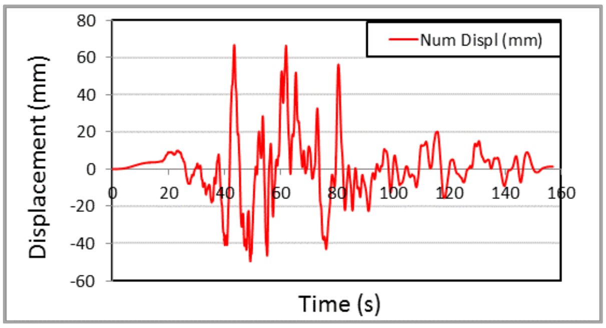

Figure 4. “Minimum” Displacement profile for 0.05g for P1524 North. [9].

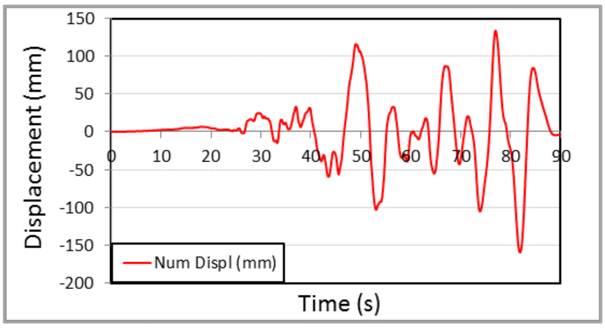

Figure 5. “Maximum” Displacement profile for 0.05g from P1155 North. [9].

A FE analysis using a minimum of 3 or 30 acceleration time histories to analyse a structure is time consuming. Besides being time consuming, it becomes difficult and confusing, even to an experienced design engineer, to select the appropriate acceleration time histories. The selection of the appropriate earthquakes can also lead to severe confusion to practicing engineers, i.e. should all the acceleration time histories conform to Fig. 2 or Fig. 3 or a combination thereof. Therefore, a more practical approach to solving this issue should be adopted. This paper therefore reviews the current approach and proposes a different approach, which is more practical to a practicing engineer.

II. Methods and Results

A balance between computational efficiency and practicality should always be enforced when conducting any analysis work. Therefore the approach identified in the EN 1998-1:2004 and other codified approaches is computationally inefficient for conducting an acceleration time history analysis of large infrastructure. The other disadvantage is that these methods select the largest value from the series of simulations as the maximum magnitude depending on the parameter. The selection of the largest value could easily be an outlier due to the spikes in the acceleration history response and therefore an unrealistic value obtained. Therefore, to limit the number of simulations, improve accuracy and efficiency, a deviations from the traditional approaches was followed.

After reviewing numerous displacement histories and observing the significant difference in the displacement profiles for a given PGA, it was decided to select acceleration time history responses which yield a minimum and maximum displacement time history response for each PGA. For ease of reference, the earthquakes which cause the smaller displacement profile will be referred to as the "minimum intensity earthquake", while the earthquakes causing the larger displacement profile will be referred to as the "maximum intensity earthquake" for each PGA. It is important to note that for each simulation, two orthogonal acceleration histories to the vertical axis should be applied to the base of the structure, i.e. in the X and Y axis, if Z is the vertical axis. Care should also be taken to apply the acceleration time histories to the structure to ensure that the earthquake loading is applied to yield the worst case scenario. The acceleration time histories should be applied so that the X-axis is oriented at 0^0, 45^0 and 90^0. Once the orientation yielding the worst case is obtained, the acceleration time histories, i.e. the minimum intensity earthquake and the maximum intensity earthquake for a specific PGA, for the remaining PGA's can easily be applied to the structure. Once the worst case orientation is established and the acceleration time histories applied, will lead to obtaining minimum and maximum envelopes for specific parameters; such as, displacements, shear forces and bending moments at specific locations within the FE model. Fig. 6 shows an example of a columns base shear response when subjected to a PGA of 0.1g.

Superimposed on the base shear force response in Fig. 6 is:

- The base shear force history as a result of the applied acceleration time history (solid red line),
- The maximum value of the base shear force (dotted red line),
- Average force, which is simply determined from the average of all the peaks (dotted blue line).
- The peak profile obtained by selecting the upper bound spikes on the base shear response (solid blue line),
- The magnitude of the peak average obtained from the average of the peak profile magnitude peaks (dotted green line). This line can be adjusted to suit the design engineers requirements based on the level of risk associated with the analysis and the structure.

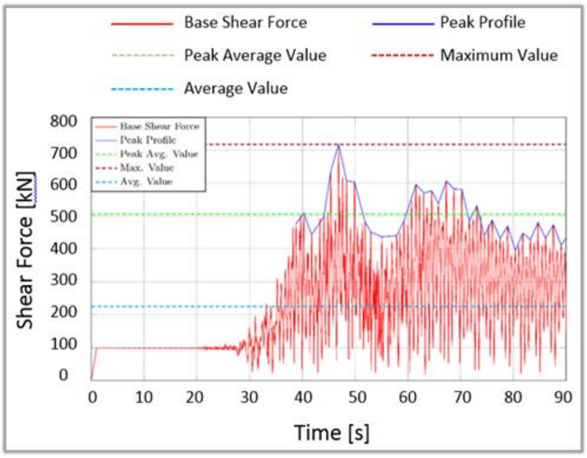

Figure 6. Example of a columns base shear response when subjected to a PGA of 0.1g. [9].

It is clear that it is incorrect to simply either use the maximum peak (dotted read line) as it is an outlier or the average of the response (dotted blue line) since it includes the minimum peaks when the average is determined. Therefore, the peak average response (green dotted line) seems the most reasonable and logical choice to use as an appropriate maximum value for the simulation. The design engineer is able to adjust the probable magnitude based on the level of uncertainty and risk associated with the project. If the maximum peak was used it would yield a maximum base shear force of 710kN compared to the peak average of 505kN. This results in a difference of approximately 41% if 505kN is used as the base value, which is significant.

This approach can now be applied to each PGA for each minimum and maximum intensity earthquake, which will result in a force band response. Fig. 7 shows the force band response of the base moments for all the columns when subjected to a 0.15g earthquake.

From Fig. 7, it clearly indicates the minimum and minimum responses which could be expected for a particular parameter of a PGA (in this case the columns' base moments). This approach will allow design engineers to apply their judgement in selecting an appropriate maximum value based on the uncertainties with respect to the parameters.

When the parameters design capacity are known it can be superimposed with the minimum and maximum responses which is shown in Fig. 8. This will clearly indicate to the design engineer whether the structure is robust to resist the forces imposed upon it due to the earthquake.

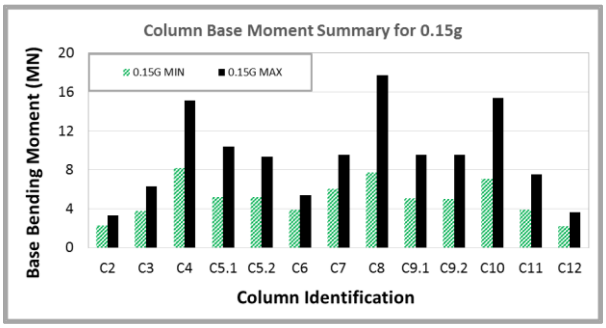

Figure 7. Summary of base moments for all columns during the Chi-Chi 0.15g earthquake.

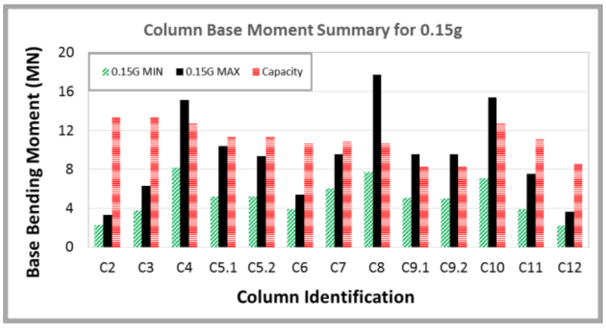

Figure 8. Summary of base moments for all columns during the Chi-Chi 0.1g earthquake.

III. Conclusion

The proposed alternative method for analysing infrastructure using the acceleration time history response was explained and developed in this paper. The alternative method is also based on using the acceleration time histories. However, it deviates from the traditional methods in that it uses a third less acceleration time histories compared to EN 1991-1:2004 and 93% less acceleration time histories compared NIST requirements. Therefore the proposed approach is computationally more efficient than the traditional methods.

The maximum parameter values from the traditional methods are based on selecting the largest value for a particular parameter. This could result in significant inaccuracies as the largest value could be an outlier due to a spike in the acceleration time history. The proposed method however uses a picking algorithm which allows the user to select either the average of the peak profile or adjusting this value to suit the practicing engineer's requirements. This approach can therefore be applied to the "minimum intensity earthquake" as well as the "maximum intensity earthquake" to obtain a force band for a particular parameter. When the design capacity is superimposed with the force band results, it provides the design engineer an immediate visual perspective of the robustness of the structure.

By using this approach it is useful when the uncertainties with regard to the maximum PGA, soil conditions, etc. are inconclusive in that the force band can be used to determine an appropriate maximum value for a particular parameter.

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