EVALUATION OF FUEL STATION CANOPY STRUCTURES SUBJECTED TO SEISMICITY

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ABSTRACT

Fuel station structures in South Africa are generally designed with wind loading as the dominant load effect, irrespective whether this infrastructure is located in seismic prone regions. In 2011, South Africa introduced a design code, specifically dedicated at determining the effect of earthquakes on infrastructure. The code however limits the maximum peak ground acceleration to 0.1g, while research indicate that certain parts of South Africa is susceptible to natural peak ground acceleration of 0.23g. From the preliminary investigations, it appears that fuel station structures are generic with respect to; member sizes, foundation sizes as well as anchor bolts lengths and anchor bolt diameters. Based on the generic fuel station structure’s properties, the changes in the code requirements, the higher expected peak ground accelerations, the economy of design and the safety of the public, an exploratory investigation was conducted to determine whether a fuel station structure is structurally robust when subjected to moderate intensity earthquakes. The investigation was conducted by subjecting a typical fuel station structure to lower and upper bond peak ground accelerations for a specific peak ground acceleration at constant intervals. This is to allow for the evaluation of lower and upper bond magnitudes against the structures design capacity. The results show that the generic fuel station structure is able to withstand a peak ground acceleration upto 0.20g.

Keywords: Fuel roof canopy structures; Seismic; FE analysis; Alternative acceleration vs time history approach

1. INTRODUCTION

Fuel station canopy structures provide cover to vehicles and fuel station attendants when refueling vehicles. These structures thus provide shelter during inclement and hot weather conditions, to allow attendants to remain dry or provide shade from the sun. These structures are designed and constructed as;

- Unbraced, where the cantilever columns support the roof canopy or
- Braced, where the roof structure is laterally braced to a building

![An unbraced fuel canopy structure.](image1)

![A braced fuel canopy structure.](image2)

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These structures, depending on the country’s seismic risk profile, can either be constructed in seismic prone or non-seismic prone regions. In South Africa, it is generally accepted that structural engineers design these structures with wind load as the dominant load case and ignore the seismicity requirements (Bastiaanse 2017, Davis 2017). Thus, if these structures are located in seismic prone regions, it is imperative that they are designed to resist the additional forces produced by earthquakes. From Figures 1a and 1b, it is observed that the columns are generally slender and carries a significant mass at its apex. As the mass of the roof canopy and / or the ground acceleration increases, it can generate significant stresses, shear forces and bending moments at the base of the columns.

The first South African seismic guidelines were published in the South Africa Bureau of Standards, SABS 0160 of 1980. These guidelines were very general and consisted of two pages. SABS 0160 of 1980 was updated in 1989 and contained more detailed seismic guidelines, which consisted of 14 pages. Most practising structural engineers did not adhere to the requirements of the code due to; not fully understanding the requirements and / or felt that the codified requirements were conservative and thus amended the partial / load factors to suit their and the client’s needs. A new code, SANS 10160-4, which is based on Eurocode 8 (2004), was introduced in 2011 and thereafter updated in 2017 (SANS 10160 – 4 : 2011 and 2017).

Fuel station structures occur throughout South Africa, however, only certain regions in South Africa are considered susceptible to natural moderate seismicity. The south-western region in the Western Cape province in South Africa is one such region. There are discrepancies on the seismic magnitude for this region. Table 1 presents the predicted seismic magnitudes for this region. Table 1 presents the predicted seismic magnitudes for this region by various authors.

<table>
<thead>
<tr>
<th>Author / Code / Guidelines</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>SABS 0160 : 1989</td>
<td>0.1g (limited to this value)</td>
</tr>
<tr>
<td>SANS 10160 : 2011 &amp; 2017</td>
<td>0.1g, but can be increased between 0.12g and 0.15g</td>
</tr>
<tr>
<td>Council of Geoscience : 2003</td>
<td>0.15g</td>
</tr>
<tr>
<td>Kijko et al : 2003</td>
<td>0.23g</td>
</tr>
<tr>
<td>Kijko et al : 2003 (Using Atkinson and Boore (2006) prediction equation)</td>
<td>0.32g</td>
</tr>
<tr>
<td>Kijko et al : 2003 (Using Jonathan (1996) prediction equation)</td>
<td>0.36g</td>
</tr>
<tr>
<td>Kijko et al : 2003 (Using Twesigomwe (1997) prediction equation)</td>
<td>0.27g</td>
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Table 1 shows the significant difference between the various author’s / codified predictions of the peak ground acceleration (PGA) for this region. The Council of Geoscience PGA map of 2003 was used to obtain the seismic risk map in SANS 10160-4 of 2011 and 2017. The codes specify a maximum PGA of 0.1g, which can be increased between 0.12g and 0.15g, using a redundancy factor (SANS 10160 – 4 : 2011 and 2017). The redundancy factor was included in the design equation to account for the reduction of the PGA magnitude (Wium, 2010). The PGA prediction of the non-codified assessments is significantly larger than the codified recommendations. Although research indicate a higher PGA, the maximum PGA is limited to 0.15g (after the redundancy modification) since South Africa is not a first world country and thus forced to reduce the PGA to maintain an economic design. This obviously comes with the risk that should the region experience a higher magnitude earthquake, significant damage could be experienced.

Irrespective of the magnitude used, earthquakes could potentially result in significant stresses, shear forces and bending moments at the base of the columns, which could exceed that obtained from the wind load condition. Based on the preceding, concerns were therefore expressed whether existing civil engineering infrastructure is able to resist the additional forces generated by these magnitude earthquakes (Solms and Haas, 2016). These concerns resulted in this exploratory investigation to
determine whether a representative existing fuel station structure is robust to resist the forces generated by a moderate intensity earthquake as well as whether the fuel station structure’s design meets the provisions of the seismic code requirements, SANS 10160 – 4: 2017.

2. METHODOLOGY

Structural engineers use various analysis techniques to analyse existing infrastructure subjected to seismicity; namely,

- Equivalent Static Analysis
- Response spectrum analysis
- Linear dynamic analysis
- Non-linear static analysis
- Non-linear dynamic analysis (Acceleration vs time history analysis)

To obtain the most accurate results, the non-linear dynamic analysis is used; i.e. the structure is subjected to acceleration vs time history responses. This method is however computationally expensive while the response of the structure is sensitive to the acceleration ground motion time history (Solms and Haas, 2016). It is therefore suggested that several acceleration vs time histories be used to obtain a realistic response of the structure. Various guidelines exist to guide structural engineers with the number of acceleration time histories to use and how to select the maximum response. These methods also assume that the PGA for the region, soil conditions at the site, soil conditions from the epicenter to the structure and degradation of the structure are known. Some of the methods; i.e. Eurocode 8 (2004) and the NIST (2011) requirements uses a minimum of 3 and 21 acceleration time history responses (Eurocode 8, 2004 and NIST, 2011). These methods also suggests that the largest value be used as the maximum irrespective if the largest value is an outlier.

To account for the uncertainties listed, the method proposed by Solms and Haas (2017), will used for this analysis. The alternative approach uses minimum and maximum acceleration time history responses for each magnitude earthquake. The reason for this approach is that two PGA’s with the same magnitude can yield significantly different displacement responses. Figures 2a and b shows the PGA histories for a 0.05g magnitude earthquake, while Figure 3a and b shows the displacement responses for the corresponding PGA’s.

Figures 3a and 3b yields significantly different displacement responses for a similar magnitude earthquake. The absolute difference in the displacements of Figures 3a and Figure 3b are 116 mm and 292 mm, respectively. This shows the significant difference in the ground motions between similar magnitude earthquakes.
The maximum value is not the largest value from the response, but is obtained from a peak picking algorithm, which can be adjusted to suit the engineer’s level of confidence as shown in Figure 4 (Haas and Solms (2017)). The results from the minimum and maximum acceleration time histories will yield a lower and upper bound response, which can then be compared to the section’s capacity. This allow for an immediate visual presentation of whether the element is likely to be damage / fail as shown in Figure 5 (Haas and Solms (2017). The reader is referred to the paper by Solms and Haas (2016) for further details on the alternative approach.
3. EARTHQUAKE SELECTION

Since SANS 10160-4 : 2017 specifies a PGA of 0.1g for the analysis of infrastructure in the south-western region of the Western Cape province in South Africa, appropriate minimum and maximum acceleration time history responses from the 1999 Chi-Chi earthquake in Taiwan were used. The minimum and maximum acceleration responses was obtained from stations, P1159 and P1468, respectively. These acceleration time responses was scaled to obtain PGA’s of 0.05g, 0.15g, 0.20g and 0.25g, respectively. The analysis was terminated when any of the parameter’s capacities were exceeded. Figures 6a and 6b presents the acceleration time history responses to obtain the lower and upper bound responses, respectively. These acceleration time history responses were also used for scaling purposes.

4. ACTUAL STRUCTURE

Figure 7 shows the actual fuel station structure, which consists of 4 cantilever rectangular hollow columns with 5 bays. The columns are 5.0 m tall and has the following properties; 350mm x 260 mm x 5 mm, with a yield strength of 300 MPa and an ultimate stress of 450 MPa. The columns are bolted to base plates with the following dimensions; 550 mm x 460 mm x 25 mm with a yield strength of 300 MPa and an ultimate stress of 450 MPa. Each base plate is bolted to the foundations using eight (8) M24 grade 8.8 bolts. The roof structure consists of a series of main and secondary beams with cladding.
5. FINITE ELEMENT MODELLING OF THE STRUCTURE

The structure was modelled to ensure a computationally efficient finite element (FE) model in Abaqus 2016. This was initially achieved by modelling the columns with beam elements and the roof canopy with shell elements. Since the stresses in the columns, the shear reactions and the tensile forces in the bolts were required, it was decided to model the columns with shell elements. Since the stresses and forces in the beams of the canopy was not required, it was decided to obtain the mass of the canopy and distribute it across the plan area of the roof with an equivalent thickness. This thus allowed the canopy structure to be modelled using shell elements. The columns, base plate and roof canopy were modelled with 4-node doubly curved shell elements with reduced integration (S4R). Figure 8 shows the FE model of the actual fuel station structure.

Eight reference points were modelled on the base plate to obtain the forces in the bolts in accordance with the actual structure. The reference points were modelled as pin connections, i.e. all translational degree of freedom (DOF) were restraint during the gravity step whereafter the orthogonal DOF perpendicular to the height was released during the earthquake motion.

To obtain a computationally efficient model, it was crucial to ensure that the mesh size vs accuracy balance was achieved. After a number of simulations, an efficient mesh size shown in Figure 8 was obtained.

Two steps were required in the simulations, namely; a static step where gravity was applied to the elements whereafter a dynamic implicit step was implemented to allow the acceleration vs time history responses to be applied to the base plates of the structure to induce motion. The FE model was checked for correctness in that it yielded the correct total vertical force (weight) in the first step, while the base
plate displacement profile matched that of the double derivative of the acceleration vs time history response. Both checks yielded accurate results to within 1% of the weight of the structure and the displacement profile of the earthquake.

The steel was modelled using non-linear material properties to allow the members to yield and not exceed the ultimate stress during the FE simulations.

6. RESULTS AND DISCUSSIONS

6.1 Yield Stresses

The actual structure was constructed using 300WA structural steel with a yield stress of 300 MPa and an ultimate stress of 450 MPa. Figure 9 presents a summary of the maximum stresses obtained for the lower and upper bound cases as obtained at the base of the columns for the various magnitude earthquakes.

![Figure 9. Maximum Lower and Upper bound stresses at the column bases.](image)

The results from Figure 9 shows that only the 0.20g Max earthquake exceeds the yield stress by 12% and thus has a probability of yielding. Since structural steel has high ductility, the material does not fail at yield but will only fail after the ultimate stress is reached. Thus, based on the results from Figure 9, we can conclude that:

- For the 0.10g Max earthquake, that the elements reach 58% of its yield stress and 38% of its ultimate stress.
- For the 0.15g Max earthquake, that the elements reach 85% of its yield stress and 57% of its ultimate stress.
- For the 0.20g Max earthquake, that the elements exceed its yield stress by 12% while only reaching 75% of its ultimate stress.

This indicates that for the severe case of a 0.20g earthquake, the structure is robust to resist the additional forces without failure.

6.2 Bolt Forces

Figure 10 shows the reference point arrangements where the baseplate is attached to the foundation through the holding down bolts. The bolts are specified on the construction drawings as M24 bolts of
grade 8.8; i.e. the bolts has a diameter of 24mm, with a yield stress of 320 MPa, a nominal tensile strength of 800 MPa, a factored tensile resistance of 225 kN / bolt and a factored shear resistance of 126 kN / bolt (SASCH, 2016).

Figures 11 and 12 presents a summary of the maximum shear and tensile bolt forces and the factored shear and tensile resistances for the 0.15g and 0.20g earthquakes.

Figure 11. Maximum bolt forces for the 0.15g earthquake
The subscribes in Figures 11 and 12 are defined as:

- \( V_U \) (X-Axis)  Shear force reaction in the X direction
- \( V_U \) (Z-Axis)  Shear force reaction in the Z direction
- \( V_r \)  Shear resistance of the bolt
- \( T_U \)  Tensile reaction in the bolt
- \( T_r \)  Tensile resistance of the bolt

Thus, based upon Figures 11 and 12 it is clear that the shear forces in both orthogonal directions are insignificant in comparison to the shear capacity of the M24 8.8 bolt’s capacity. The maximum shear forces in both orthogonal directions to the vertical only attains 10% of the bolts shear resistance, thus indicating no possibility of bolt shearing.

Also, based on Figures 11 and 12, the bolts tensile forces only reaches 76% of the tensile capacity of the M24 8.8 bolt’s capacity. Thus, this indicates that the bolts are unlikely to fail in tension.

6. CONCLUSION

The fuel station structure was analyzed using the approach suggested by Solms and Haas (2016). This approach uses a minimum and maximum earthquake response history to obtain a lower and upper bound force response. The lower and upper bound forces are compared to the actual capacity / yield stress / ultimate stress of the elements.

Based on the FE simulation results it can be assumed that;

- The wind load is the dominant load case since the forces obtained from the FE simulations are significantly less than the elements capacities, or
- The member sizes were arbitrarily increased since the structures are generic and thus to avoid continuous analysis.

The results also indicate that the structure can easily sustain a moderate intensity earthquake, which meets codified requirements upto 0.20g. From this single investigation, it cannot concluded that all fuel station structures meet the SANS 10160 – 4 : 2017 requirements. It is important that more fuel station structures of various shapes and sizes be analyzed, in order to make a conclusive recommendation on this matter.
7. REFERENCES

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