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Friction Dampers for the Seismic Control of an Underground Paste Tank in Indonesia

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Abstract

The structural design of an underground 20m tall heavy steel structure was found to have reactions at the base which were of significant magnitude to connect for. This structure supports two 15 m tall 300 ton cement silos along with process equipment including mixers, hoppers, and pump units. The structure will be used as an underground paste backfill production plant. The plant is owned and operated by the mining company *PT Freeport* and is located in Irian Jaya (West Papua), Indonesia. Its function is to convert the waste product from mining operations into a "paste" form by grinding tailings and mixing the tailings with a binder in order to create a material that can be pumped into underground mine voids and stopes. The process for the paste plant was provided by *Golder Paste Technology Ltd.*

The structure is located in an extremely active seismic zone, calling for stringent earthquake-resistant design considerations. The two supported silos each have a capacity of 300 tons of material and are therefore potentially solicited to very large inertial forces during a seismic event. The large size and proximity of the two silos also require displacement-control measures in order to avoid the possibility of the two silos colliding with each other. The option to attach the top of the structure to the walls of the cavern in which the structure will be located was examined in order to reduce the overturning moment of the structure, the base shear, and the magnitude of forces in the seismic force resisting system (SFRS). This top connection also limits the sway motion of the silos and reduces the risk of the silos colliding with each other. However, such an external connection would increase the structure's response to a seismic event due to its increased stiffness. Small scale models were developed and analyzed in order to determine if attaching a structure externally at a point along its height is beneficial to a structure, or, if the added stiffness of the structure will attract too much additional seismic load to the point where it is detrimental to the structure. Friction dampers were used as an energy dissipating mechanism at the top external support. The Friction dampers essentially act as rigid supports to a certain predetermined "slip force" before beginning to dissipate energy to the cavern walls once this threshold is surpassed. The force threshold for the friction dampers was selected such that the structure would remain in the elastic range while most of the energy would be dissipated through the friction dampers to the cavern walls. A full scale three dimensional model was then analyzed and compared to the smaller scale models to verify if the conclusions were coherent.

Keywords: Friction dampers, Seismic design, Time-history analysis, Energy dissipation.

Introduction

The design of an underground 20 m tall heavy steel structure which supports two 15 m tall 300 ton cement silos along with process equipment including mixers, hoppers, and pump units for an underground paste backfill production plant. The plant is owned and operated by the mining company *PT Freeport* and is located in Irian Jaya (West Papua), Indonesia. Its function is to convert the waste product from mining operations into a "paste" form by grinding tailings and mixing the tailings with a binder in order to create a material that can be pumped into underground mine voids and stopes. The process for the paste plant was provided by *Golder Paste Technology Ltd.*

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The structure in question is to be built in West Papua, Indonesia, which is an area with much seismic activity. The initial design of this structure was carried out as the structure being free-standing. With the free-standing structure, the reactions at the base were subsequently too large and the connections required at the base were not feasible. At this point, the option of connecting the structure to the walls of the cavern within which the structure is located was suggested. Due to the proximity of the cavern walls to the exterior perimeter of the structure this option was geometrically feasible, however, it remained to be determined if this option was structurally viable.

The main idea was to give this structure additional external supports at the main uppermost level where the silos are connected to the supporting structure beneath. This was expected to have both positive and negative effects. The presumption was that the base shear would be reduced due to the presence of the additional supports, and the overturning moment would also be reduced due to the location of these new external supports. However, theoretically these rigid supports being located at an elevation higher than the base, should increase the stiffness of the structure. By increasing the stiffness of the structure, the period of the structure would decrease and this would in turn, increase the structure's response to a seismic event. The now more rigid structure would experience higher seismic loads due to it's increased excitation to an earthquake. What needed to be determined was whether it would an option to connect the structure at it's top, or whether this would have more negative than positive effects.

The risk that the rigid supports near the top of the structure, during an extreme seismic event, would transfer very large forces to the structure was high. For this reason, the use of friction dampers at these supports was introduced to limit the load that could be transferred to the structure up to a specific predetermined limit. The use of friction dampers at these locations, by acting as fuse elements, and not allowing increasingly larger loads to be transmitted during an extreme seismic event would eliminate the risk of unfavorable resonance occurring. The frictions dampers selected would be designed to act as a rigid support as long as a specified force referred to as the "slip force" was not exceeded. If this specific "slip force" is ever exceeded, the damper would slip within itself and dissipate any additional energy, acting analogously to a braking system on a vehicle. This slip mechanism would ensure that the structure does not experience any load greater than that of the specified slip force.

Small scale model tests

Different scale model tests were first carried out to verify the effect that connecting a structure's top to an external support would have on the loading experienced by the structure. The initial goal was to determine how the base shear and overturning moment would be affected with or without the top connection. Different pairs of models were examined and different variables introduced in each case. Each model was subject to a Response spectrum. The Response spectrum used for the small scale model tests has spectral accelerations of 0.69 g, 0.34 g, 0.14g, and 0.48 g for periods of 0.2 s, 0.5 s, 1.0 s, and 2.0 s respectively. This response spectrum is the response spectrum for Montreal, Quebec, Canada. This response spectrum was selected because Montreal is a relatively high seismic zone. The response spectrum was consistently applied to each test model.

Before modeling any damper in great detail, it was concluded that modeling the damper as a rigid external connection which would have only its horizontal degree of freedom restricted would be adequate. As long as the slip load of the friction dampers is larger than the external reaction at this damper, this assumption is valid.

The first pair of structures to be examined were modeled in a manner to ensure that both the free-standing structure and the structure which was fixed at it's top would have all periods for all of their mode shapes that are less that 0.2 s to ensure that the added stiffness from the top would not effect the structures response to a seismic event. The test models consisted of a four storey two-dimensional concentrically braced frame with storey heights of 3 m and bay widths of 5 m. The column and beam sections, arbitrarily selected, were W310 x 97 and all the brace members were selected as HSS 152 x 152 x 8.0mm. Lastly, each joint was given a joint mass of 102kg (load of 1000 N) for a total load of 2000 N per storey, or 8000 N for the entire structure. The models 1 and 2 can be seen in figure 1 below.

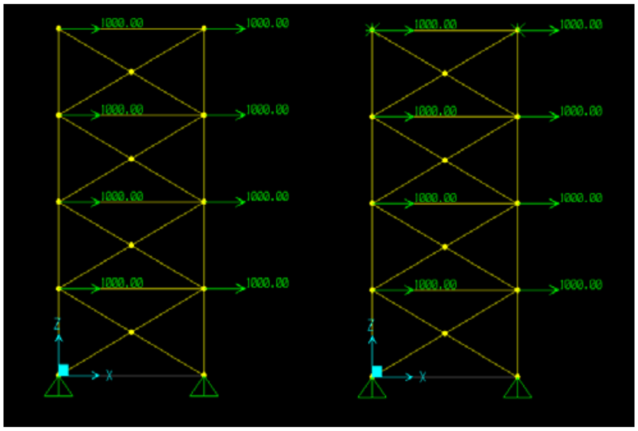

Figure 1 - Model 1 and Model 2 (Stiff Models)

The analysis results for the comparison of model 1 and model 2 can be found in Table 1 below.

Table 1 - Comparison of Model 1 and Model 2

Model #	1. Free-Standing structure	2. Structure Connected at Top
Period of first mode shape	0.090 s	0.035 s
Period of Second mode shape	0.028 s	0.017 s
Period of third mode shape	0.015 s	0.015 s
Lateral force at base of structure	74.72 kN	40.04 kN
Lateral force at Top Connection	N / A	28.10 kN
Total Lateral Load	74.72 kN	68.14 kN
Maximum Vertical Force at column base	132.36 kN	12.78 kN

The total lateral forces in each case were very similar. The total lateral force of the second model (68.14 kN) was slightly smaller than the total lateral force of model 1. This difference in total lateral force is due to the fixed supports being inline with the mass at the top level of model 2, and thus prevent those masses from accelerating, which in turn reduces the total lateral force. It should be noted that the Lateral Force at the Base was reduced by 46%. If we use the vertical force at a base support to represent the overturning moment, the overturning moment is reduced by 91%. It is important to note that the periods for the first three mode shapes for each model are all less than 0.2 seconds and for this reason the relative stiffnesses between the two support conditions does not change how each model is excited by the response spectrum selected. Model 2 has a higher stiffness than the free-standing model. This is evident due to the Model 1’s first mode period being greater than Model 2’s first period.

The next pair of structures to be analyzed were structures modeled such that by changing the support conditions, would render a flexible structure stiff. In these two cases, the total lateral forces are not expected to be similar to each other. In fact, the model with additional supports is expected to have higher total lateral forces due to its increased stiffness which should respond more to a seismic event.

All the member sections (columns, beams and braces) were kept the same as the previous two models, as well as the height of each story and the width of each bay. The number of stories was increased from four to twelve. This exercise was performed in order to ensure that the free-standing model (model 3) was flexible enough to have at least it's first mode period greater than 0.2 seconds. The same mass as in the previous two models was assigned to each node. Model 3 and model 4 can be seen in Figure 2.

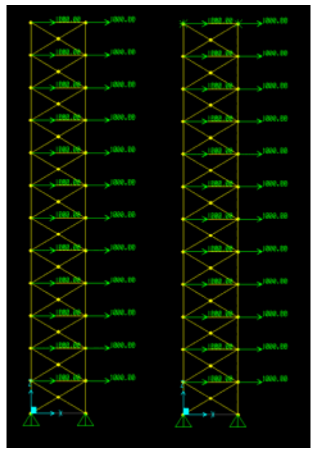

Figure 2 - Model 3 and Model 4 (Flexible structures)

The analysis results for the comparison of model 3 and model 4 can be found in Table 2 below.

Table 2 - Comparison of Model 3 and Model 4

Model #	3. Free-Standing	4. Connected at Top
Period of first mode shape	0.512 s	0.144 s
Period of Second mode shape	0.110 s	0.059 s
Period of third mode shape	0.051 s	0.045 s
Lateral force at base of structure	111.2 kN	134.4 kN
Lateral force at Top Connection	N / A	85.3 kN
Total Lateral Load	111.2 kN	219.7 kN
Maximum Vertical Force at column base	484.5 kN	217.4 kN

The total lateral force in model 4 was much larger than the total lateral force from model 3. The period of the first mode shape for model 3 was 0.51 seconds. At a period of 0.51 seconds the flexibility of a structure is large enough to start responding less to a seismic event. Model 4 has a period which is small enough to be on the "plateau" of small structural periods (0.20 seconds) and is therefore excited by a seismic event more than model 3. When a comparison is made between models 3 and 4, the fixed model experiences an increase in total lateral force by approximately 98% and an increase in lateral load at the base of approximately 21%. In view of this fact alone, it may seem like a disadvantage to fix any structure anywhere along its height. However, when the overturning moment is examined, the reduction in force is apparent. By fixing the model at its top, the overturning moment is reduced by approximately 55 %.

It can be concluded from the analysis of the two pairs of models presented thus far, that as the stiffness of a structure increases, the use of a fixed support at a given height in the structure will benefit the structure by reducing the total lateral load, the base shear, and the overturning moment. It may appear that for structures that are relatively flexible, the use of fixed supports at a given height may not be worthwhile due to the increase in total lateral loads and consequently the increase in base shear. Despite the lateral load and the base shear being larger, the local forces in each of the members may be lower with a fixed support and therefore the overall weight of the structure may decrease. This should be examined on a case-by-case basis.

Results from models in-between the four-storey and the twelve-storey models were then analyzed, keeping all the same member sections as previously mentioned. One storey at a time was added to the four-storey model. In total, nine pairs of models were analyzed in order to examine what the behavior is between fixed and free frames ranging between four and twelve stories. The percentage change between the free-standing model and the fixed model were compared for each frame plotted against the period of the first mode of the free-standing tower. A positive percentage indicates that the variable increases when a support at the top of the tower is added. The opposite is true for a negative percentage. The graphs below include a range of periods found in most common structures. The percentage change when fixing the top of this model is reported for lateral load at the base, total lateral load, and overturning moment.

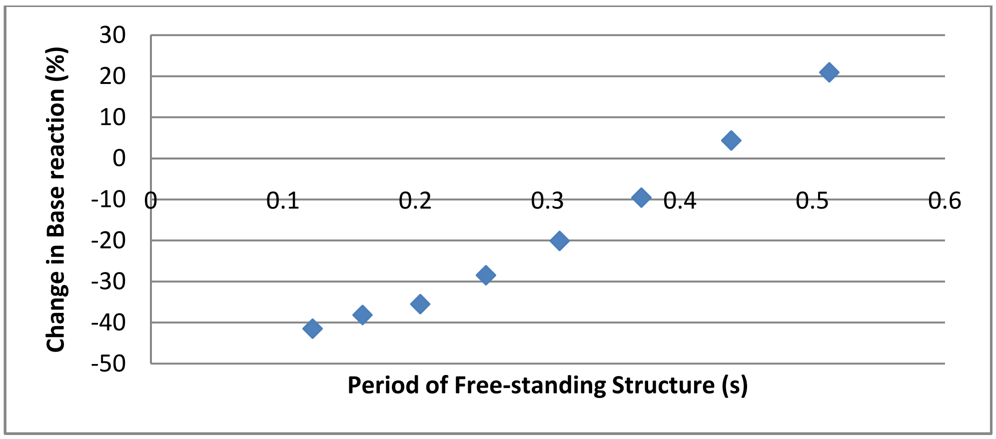

Figure 3 - Change in reaction at base between a free-standing structure and a structure fixed at its top vs. the period of the free-standing structure

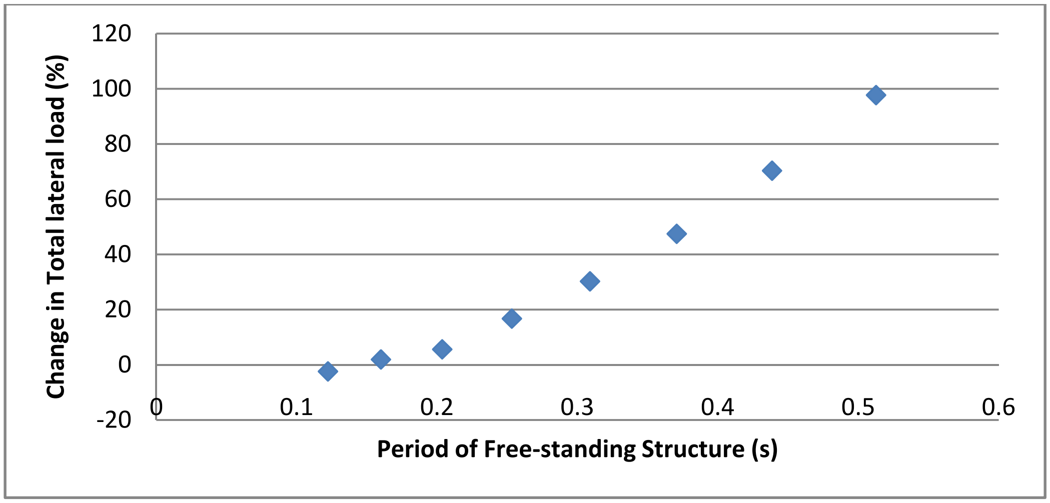

Figure 4 - Change in total lateral load between a free-standing structure and a structure fixed at its top vs. the period of the free-standing structure

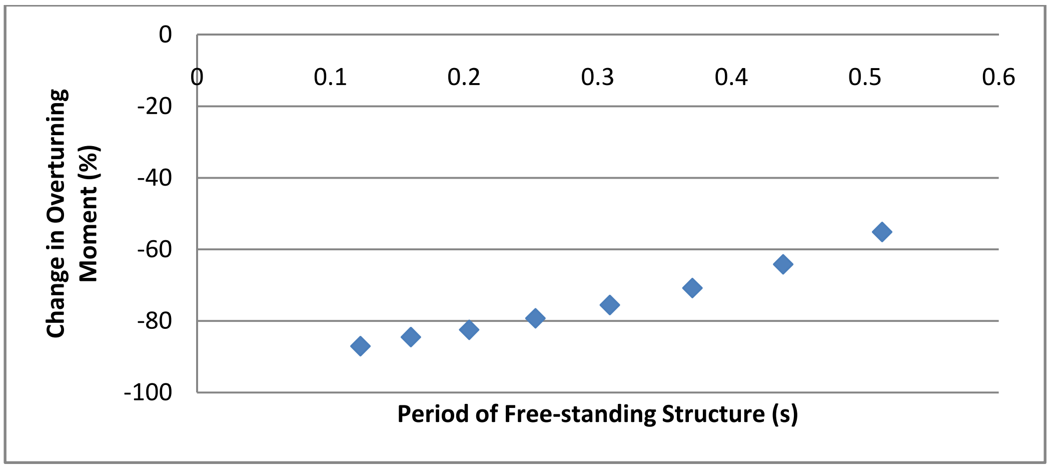

Figure 5 - Change in Overturning Moment between a free-standing structure and a structure fixed at its top vs. the period of the free-standing structure

Structures externally supported at mid-height

Structure may, for whatever reason, not be able to be supported at their peak. For this reason some structures may have an external support at some fraction of their total height. For the case of the silo structure which is to be located in Indonesia, the very top of the structure could not be connected to the external walls of the cavern. The silos, which alone make up approximately half the height of the structure, are non-structural components and therefore were left free-standing. The top of the steel frame was the selected as the location for the external lateral supports (the dampers).

Three new pairs of models were compared to study the reductions in lateral load at the base, total lateral load, and overturning moment when a free-standing structure would be externally supported at its mid-height. Because the structure in Indonesia will have the majority of its mass located above the external brace, contained in the silos, larger masses were assigned to the nodes above mid-height in the model than below mid-height nodes. The first of the three models externally connected at mid height had an equal mass at every node. The second structure externally connected at mid height had the mass assigned to each node above the external support be twice as large as the mass assigned to the nodes below the mid-height. The third structure externally connected at mid height had the mass assigned at the nodes above the external support five times as large as the mass at the nodes below the mid-height. The analysis results are found in Figures 6 – 9.

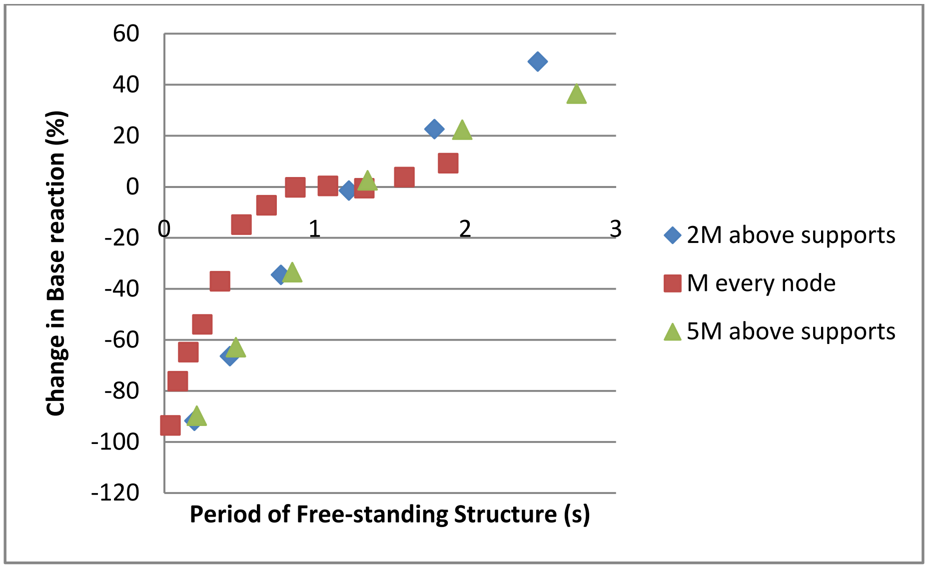

Figure 6 - Change in reaction at base between a free-standing structure and a structure fixed at its mid-height vs. the period of the free-standing structure

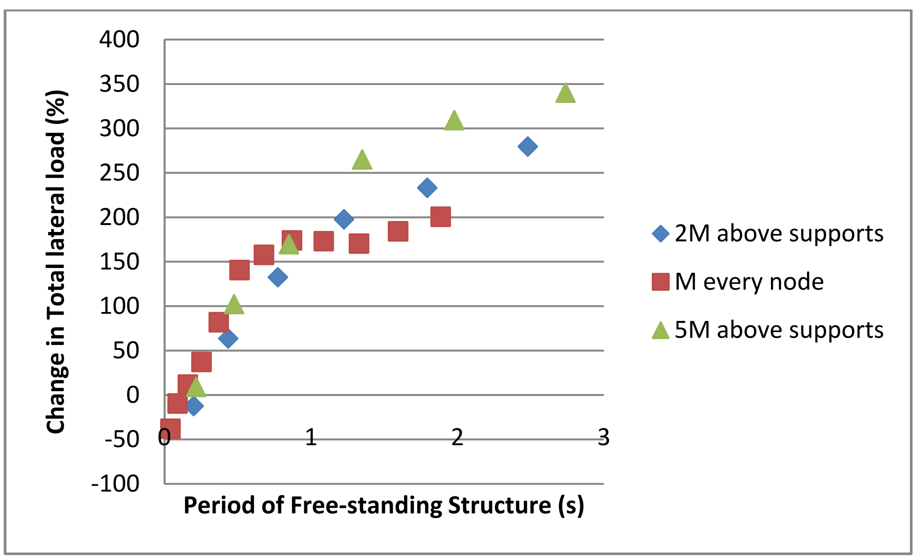

Figure 7 - Change in total lateral load between a free-standing structure and a structure fixed at its mid-height vs. the period of the free-standing structure

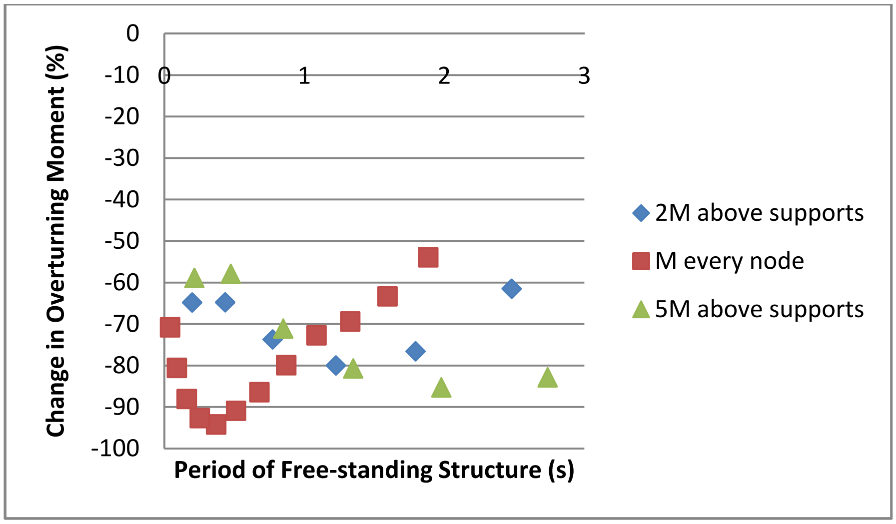

Figure 8 - Change in Overturning Moment between a free-standing structure and a structure fixed at its mid-height vs. the period of the free-standing structure

Application of our findings to the tower located in Indonesia

The concentrically braced structure which is to be located in Indonesia is made up of a main tower comprised of three stories with a total height of approximately 20 m which in turn supports two 15 m tall 300 ton silos used to store materials from mining operations. From this basic information the flexibility of the structure was required in order to determine if this structure is stiff enough to consider using a fixed support at its mid-height, or if this action would be detrimental to the structure. Empirical formulas to approximate the period of a structure based on limited information can be used as a first approximation. In this case, the empirical equation as suggested in the 2005 *NBCC* for braced frames was used to obtain an approximation of the structure's period. With a height of the structure of 20 m the empirical formula:

$$T = 0.025\, h_n \qquad (1)$$

gave a structural period of 0.5 s. From the figures 6, 7, and 8 above, the lateral load at the base as well as the overturning moment would decrease and total lateral load would increase by adding fixed supports at approximately midpoint if it was taken into account that the majority of the weight is located above the external supports at mid-height. The full scale model can be seen in figure 9.

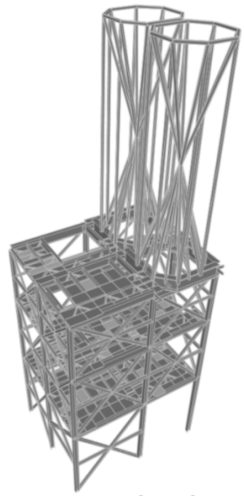

Figure 9 – View of the full scale Model

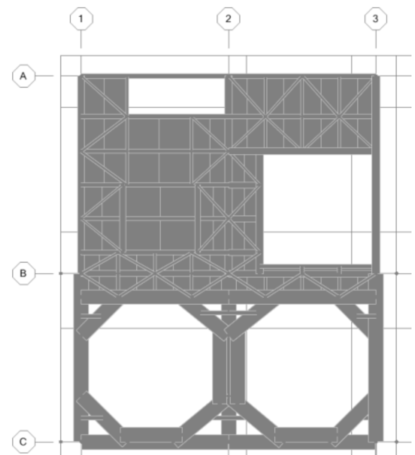

Figure 10 – Plan view of full scale model at elev. 20 m

The model was created, analyzed, and designed with the assumption that a fixed structure would be used rather than a free-standing tower. The Dampers which will connect the structure to the cavern walls were not yet designed. The design of the dampers will occur once the required slip load was determined from the external reactions expected at these supports. The dampers were modeled simply as horizontal tension-compression members which connect a node in the structure to an external support. This support would restrict any displacement in the longitudinal direction of the member only. These fixed supports were applied at seven locations at an elevation of 20 m above the base of the structure. This location is at the top of the steel frame, which is at the base of the two 15 m silos. A plan view of this level with the seven external supports can be seen in Figure 10. The octagonal shaped openings seen are the bases of the silos.

The structure was analyzed with the fixed supports in place. The same response spectrum as previously used for the test models was applied to the fixed model and member selection was carried out in order to give the structure a more accurate stiffness and therefore amore accurate seismic response. The external supports were then removed and the model was permitted to be free-standing. The members were kept the same as for the fixed structure in order to make the only variable the fixed or free-standing condition. Table 3 shows a summary of the comparison between the two models.

Table 3 - Comparison of fixed and free towers subject to a Response Spectrum analysis

Model	Free-Standing	Connected at Top
Period of first mode shape	1.034 s	0.548 s
Period of Second mode shape	0.904 s	0.513 s
Period of third mode shape	0.533 s	0.505 s
Lateral force at base of structure	1824 kN	895 kN
Lateral force at Top Connection	N / A	3881 kN
Total Lateral Load	1824 kN	4776 kN
Maximum Vertical Force at column base	2870 kN	1058 kN

The base shear when the structure was connected at its peak was reduced by 51% and the overturning moment was reduced by approximately 63%. It was found that the free-standing structure is more flexible and therefore attracts less lateral load, however the free-standing structure has a greater base shear. This would most likely require the free-standing structure to have larger bracing elements then the elements with which it was analyzed, which would in turn decrease its flexibility and increase the lateral load of the free-standing structure. It should be noted that the table above provides lower bound values for the forces on the free-standing structure. The results from the full scale model differed slightly from Figures 6, 7 and 8, however they were still quite close to the values which can be interpolated from the these figures. The slight discrepancy could be caused by the fact that the model is a full scale three-dimensional model. This 3-D model included higher mode shapes and effects from both horizontal directions which were not present in the 2-D small scale test models.

A time-history analysis was also carried out on the full scale structure to ensure the comparison between the two connection conditions would again agree with the results stated above. In order to analyze the free-standing structure against the fixed structure, an arbitrary time-history was selected. The time-history "24538-S2486-94020.06 SANTA MONICA - CITY HALL GROUNDS" from SAP 2000's default time-history functions was used. A linear time-history analysis was performed on the two towers. The results can be seen in Table 4.

Table 4 - Comparison of fixed and free towers subject to a Time History analysis

Model	Free-Standing	Connected at Top
Lateral force at base of structure	4671 kN	1496 kN
Lateral force at Top Connection	N / A	5259 kN
Total Lateral Load	4671 kN	6755 kN
Maximum Vertical Force at column base	7789 kN	1792 kN

The time-history analysis results are consistent with the response spectrum results. Once again the base shear is reduced by over fifty percent, as well as the maximum vertical reaction at any column. The total lateral load once again increased with an increase in stiffness of the model (fixed model).

Conclusion

The time period for the structure to be located in Indonesia is low enough, and therefore the structure is stiff enough, to warrant the use of external supports at approximately mid-height of the structure. As predicted by the small scale models, the full scale model saw its lateral load at the base and overturning moments decrease due to the implementation of external supports. It is not in every case where attaching the top or the mid-height of a structure to an external support is advantageous. By referring to the graphs provided for the different external connection conditions, and if the structural period of the structure is known, it can be quickly determined if external supports will reduce or increase the lateral load at the base, the total lateral load, and the overturning moment.

References

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