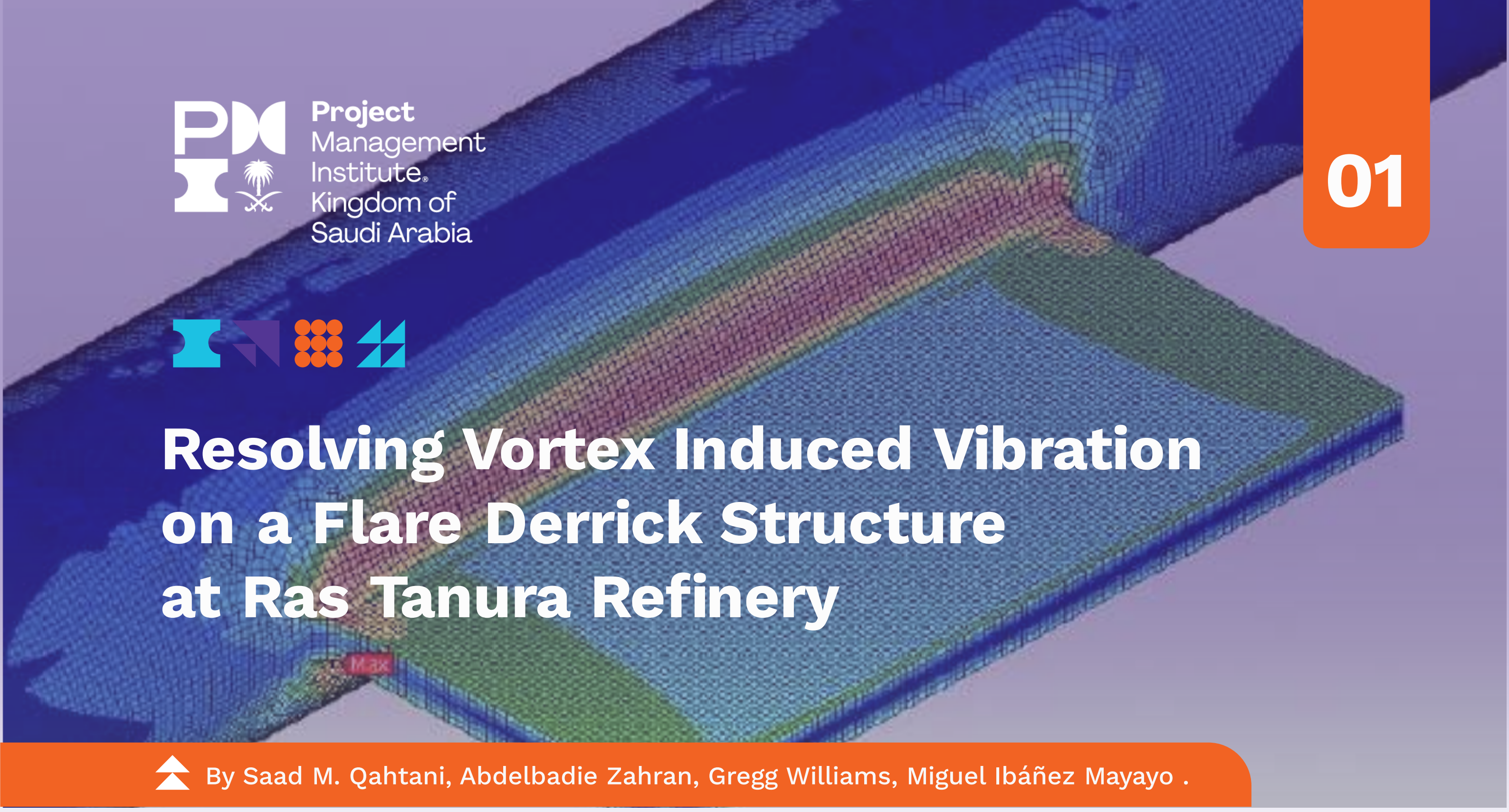


Resolving Vortex Induced Vibration on a Flare Derrick Structure at Ras Tanura Refinery

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Abstract

A 120 m high derrick structure, composed of tubular structural members, was found to have a detached gusset plate in the field and a study was launched to determine the root cause of the detachment.

The analysis determined that the failure was due to vortex-induced vibration of the structural pipe member.

After determination of the root cause, a further study was conducted to verify the effect of vortexinduced vibration for similar structures, utilizing the same design philosophy, to determine the expected deflection and resulting stress on the gusset plates and welds, which could lead to therecorded detachment.

An engineering and construction study was launched to determine how best to implement the proposed solution and obtain engineering robustness in an existing structure, taking into account project constraints such as schedule, cost, and accessibility. Finally, the solution implemented was based on structural mass dampening, which utilizes a specifically calculated additional mass, clamped to the member(s) determined to be prone to vortex-induced vibration. This method increases the structural damping (Scruton number) of the affected members and reduces the amplitude of expected vibration to levels that theoretically provide infinite fatigue life. This solution offered a lightweight, easy-to-install solution without the need for invasive structural modifications or welding. The mass dampeners were installed via rope access rather than more traditional methods of structural steel installation.

In 2020, three years after the erection and assembly of the 120 m derrick-supported demountable flare system, a field report was received showing two internal support members detached from the main derrick structure. The report contained several photos taken from the ground, indicating the two detached members, which are shown in Figure 1.

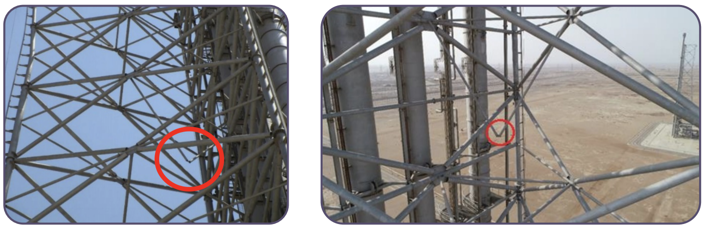


Fig. 1 Photos of the detached members.

Introduction / Original Findings

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

The damage to the 120 m tall derrick structure was of particular concern, since in 2020 a similar derrick-supported demountable flare system at Aramco's Ras Tanura Refinery had been installed. The Ras Tanura flare system is 200 m tall with five flare risers (and future space for three additional risers) supported by a 190 m tall derrick. The derrick was successfully erected between 2019-2020.

Due to the size of the Ras Tanura derrick system, and the similarities in the design philosophy to the 120 m tall derrick, which experienced detachment, an additional precautionary study was conducted to determine whether the same field detachmen- could be expected at Ras Tanura.

While the two projects had many design similarities, the size of the Ras Tanura derrick was much larger. Although this resulted in a design with additional columns and internal bracing, to reduce the overall span of the bracing members, it also meant that each level of the derrick had up to 12 unbraced simply supported members.

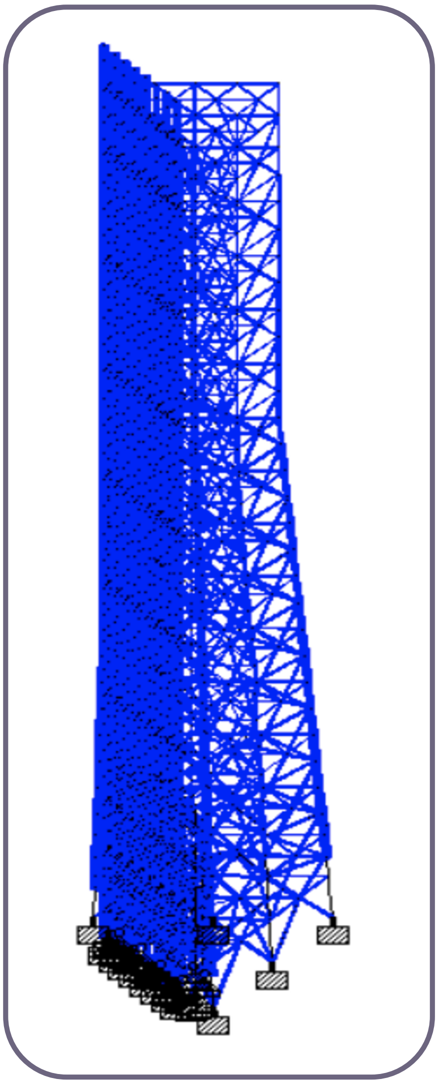


Fig. 2 3D View of the flare derrick for the Ras Tanura Project

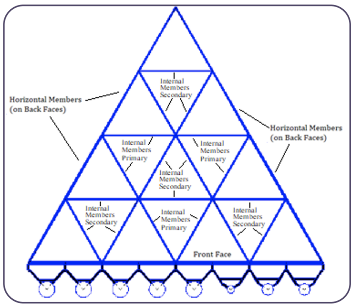
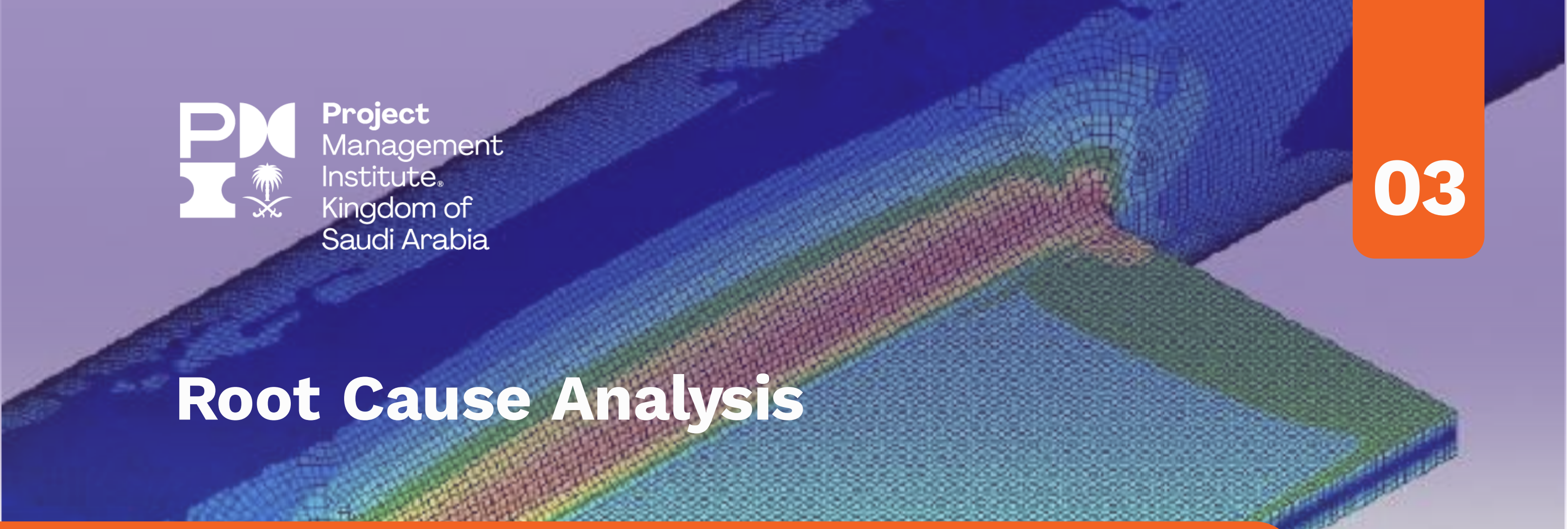


Fig. 3 Plan view of the derrick structure.



Root Cause Analysis

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

The original structural design was reviewed, and it was confirmed that the derrick structure met the design wind conditions per design code (2005 ASCE-7) (1) as required by the project specifications. Based on this analysis, it did not seem that a deficiency in the static strength of the members or gusset plates would lead to the detachment found in the field. Poor weld quality was also considered as a potential cause of the detachment. Due to the ongoing production at the site and the elevated position of the detachment, detailed assessment and in-situ NDT of the joint were prevented. Visual inspection showed localized cracking and visible vibration at low wind speed was observed. A study of existing quality documentation did not provide any evidence to suggest a lack of quality of the weld on the member.

As no quality or fabrication deficiency was evident, other options were investigated to determine the root cause. The risk of vortex-induced vibration (VIV) of individual members was investigated due to the observed vibration and eventually was considered as the primary root cause. Research into the risk of vortex-induced vibration highlighted the following key points:

- VIV is a complex aero-elastic behavior and design criteria based on mature theory is not available for individual members in bolted structures.
- Most evidence of VIV are found on slender structures with enclosed (usually circular) cross section. The vibration is transverse to flow direction.
- Steady upstream flow is essential for vortex shedding to develop. Vortex shedding tends to occur with steady continuous winds at a critical velocity.
- Winds which potentially cause vortex shedding are steady winds with velocity between 5 to 15 m/s. To sustain the vibration, the flow should be laminar and constant, but exact definition of flow type and duration to cause vibration is unknown.
- Gusty winds, such as wind in a severe storm, typically will not cause vortex shedding. In fact, if the wind velocity is greater than 15 m/s (35 mph), the wind is generally too turbulent for vortex shedding to develop.

Root Cause Analysis

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Many international design codes address vortex-induced vibrations for global systems, such as chimneys, welded towers, and bolted towers, but the design codes generally do not provide much guidance on local effects within a structure. The Architectural Institute of Japan (AIJ) - Stack Structural Design and Construction Code, 1982, (2) provided some criteria for truss towermembers to avoid vortex shedding vibration.

Applying the methodology as per the AIJ (2) showed that vibration could be induced on internal horizontal members, including the two detached members on the damaged structure. When applied to the Ras Tanura derrick, up to 168 internal members were shown to be prone to vibration.

The limitation of the AIJ methodology is that it does not define the significance of the vibration or if the vibration will cause fatigue failure. Therefore, the AIJ methodology was used to determine which members could vibrate, and then a deeper analysis was used to determine the fatigue stress to be expected in any member.

Engineering Study Using AIJ Methodology

The starting point for the technical study was to use the methodology as outlined in the AIJ code. This required calculating the resonant velocity for each fully exposed derrick member usingfollowing formula and procedure:

$$Vr = \frac{D\,f}{S}$$

Vr: resonant velocity of the individual members [m/s]
D: width of the individual members [m]
f : 1st natural frequency of the individual members [Hz]
S: Strouhal number (0.18 for steel pipe)

The resonant velocity (Vr), shall be higher than the line of Figure 4, depending on the height above ground. (i.e., 15~20 [m/s] or higher).

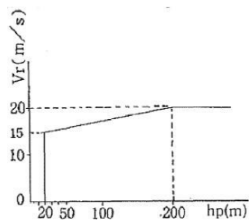


Fig. 4 Height vs. resonant velocity (Vr)

The following acceptance criteria are checked for tubular derrick members – the resonant velocity of the individual member calculated shall be more than resonant velocity for member's elevation obtained from Figure 4. Members meeting this condition are not prone to large vibrations. For those members that failed to meet the condition, a risk of vibration was identified, and an additional analytical approach was applied to determine the vibration amplitude and induced stress in the end connections.

Project Management Institute. Kingdom of Saudi Arabia

Engineering Study using Analytical Approach Methodology

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

To reduce the likelihood of vibration of slender members on the derrick, "Flow-Induced Vibration," 2nd Edition, by Robert D. Blevins (3), is used as a reference for methodology to determine the deflection amplitude and moment caused by the vibration of the beams. When a member vibrates, it will reach a certain amplitude of deflection at various points of the beam depending on the mode of vibration. For the first mode of vibration, this amplitude will be at the midpoint of the beam. This deflection can then be used to determine a moment that is transmitted to each connection plate, where failures of the beam would be found. Table 3-2 in Blevins (1990) (3)provides four separate expressions for determining the predicted displacement amplitude during vibration. Of the four methods for predicting the displacement amplitude, the fourth method, the Harmonic Model method, consistently provides the largest amplitude, so this method is used to develop an expected deflection at the mid-point of the beams in the Ras Tanura derrick structure. The formula provided is as follows:

$$Ay = \frac{CL \times D'}{4 \times \pi \times S^2 \times \delta_r}$$

Where:

Ay = Max. resonant amplitude

CL = Lift coefficient

D' = Avg. diameter perpendicular to flow

S = Strouhal number (taken as 0.18)

δr = Scruton number

Per Table 3-2, the Life Coefficient is given by Table 3-1 in Blevins. For a sinusoidal deflected shape, this factor is given by the following function:

$$CL = a \times \left(\frac{\pi^2 \times l}{8 \times L}\right)^{0.5}$$

Where:

a = Calculation Factor equal to 0.35

l = Length of sinusoidal section

L= Length of pipe

Substituting the maximum value l/L leads to the following values for CL:

$$CL = 0.35 \times \left(\frac{\pi}{\sqrt{8}}\right) = 0.389$$

For each pipe member, the Strouhal number, the diameter, and the lift coefficient remain constant, and therefore cannot be changed to alter the expected displacement of the beam during vibration. The other variable used to calculate the deflection amplitude is the Scruton number, also known as the structural damping value, which per Table 3-2 is given by equation 3-12, provided below:

$$\delta_r = \frac{2 \times m_e \times (2 \times \pi \times \zeta)}{\rho \times D'^2}$$

Where:

me = Equivalent linear mass of the beam

ζ = Structural damping ratio

ρ = Density of air (taken as 1.18kg/m3)

D'= Avg. assembly diameter perpendicular to flow

Engineering Study using Analytical Approach Methodology

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

It should be noted that the structural damping ratio is related to the logarithmic decrement value (ξ) by a factor of 2π, and therefore 2πζ can also be replaced by ξ. Per Section 3.6 in Blevins (3), the Scruton number should be kept above a value of 64. In application, this can be difficult to achieve for slender members. Other references for cylindrical members, such as CICIND Model Code for steel chimneys (4), note that for Scruton numbers less than 5, vibration amplitudes may be violent. For Scruton numbers greater than 25, vibration amplitudes may be minimal, and for Scruton numbers in between, the cylindrical part should be stabilized/damped; or the stresses should be verified to fall within threshold limits. Because the Scruton number is in the denominator of the deflection amplitude equation, the impact of the Scruton number is greater at lower values, while higher values of the Scruton number do not have as much impact on the deflection.

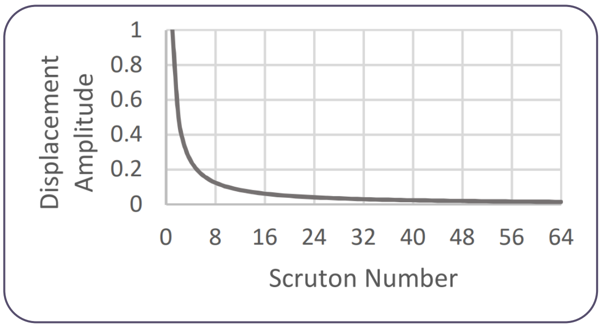


Fig. 5 – Normalized deflection amplitude as a function of the Scruton number.

The damping value (logarithmic damping) for structural members is not well documented in international codes and standards. Structural damping values for items like chimneys and stacks are well documented, but values for simple beams are not readily available. Preliminary reviews of members had used a logarithmic damping ratio of 1%. Further Analysis based on videos from site determined that a conservative estimate for logarithmic damping would be 0.5%. This value represents a damping ratio of 0.08%.

Fatigue Analysis

Based on the geometry of the gusset plates and weld joints, these parts would be classified as Case 1.2 and Case 5.6 of AISC 360-10 (5) Table A-3.1, respectively (as seen in the snapshots below). Between the gusset plate and the weld, the weld stresses were determined to be the governing aspect of any design failures. Table A-3.1 table provides a threshold fatigue stress of 110MPa and 69MPa, respectively. Therefore, if the stress in the gusset plates and weld were kept below this threshold, no fatigue issues were expected to be found.

Fatigue Analysis

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Case 5.6 from AISC 360-10 (5) was found most applicable for two reasons:

- An FE study was conducted to determine where the highest stress around the weld joint would be found, between the root, throat, and toe of the weld. This analysis found that the highest stresses are found in the toe of the weld.

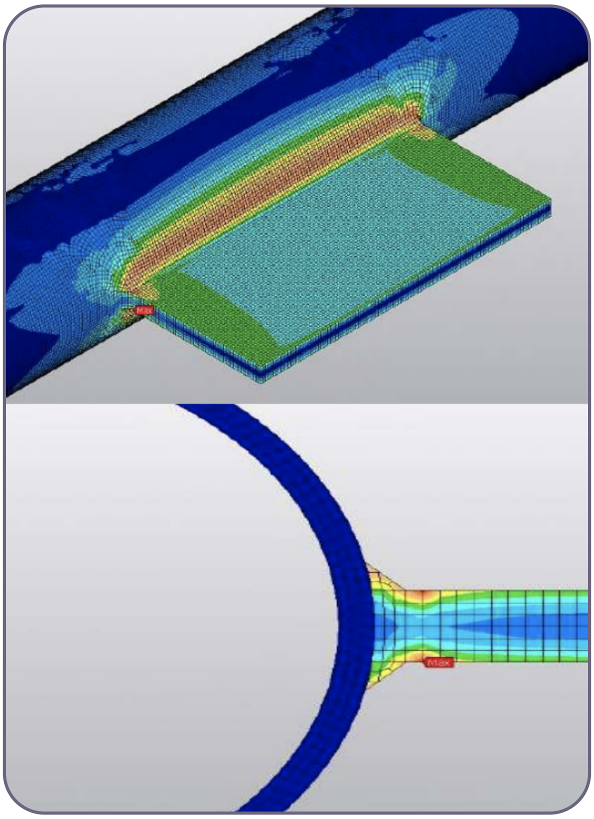


Fig. 6 – Images from FE Study

- Results from published testing of T joints under bending have higher threshold stresses compared to joints loaded only in tension.

Possible Solutions

Some recognized approaches to alleviate the effect of VIV are:

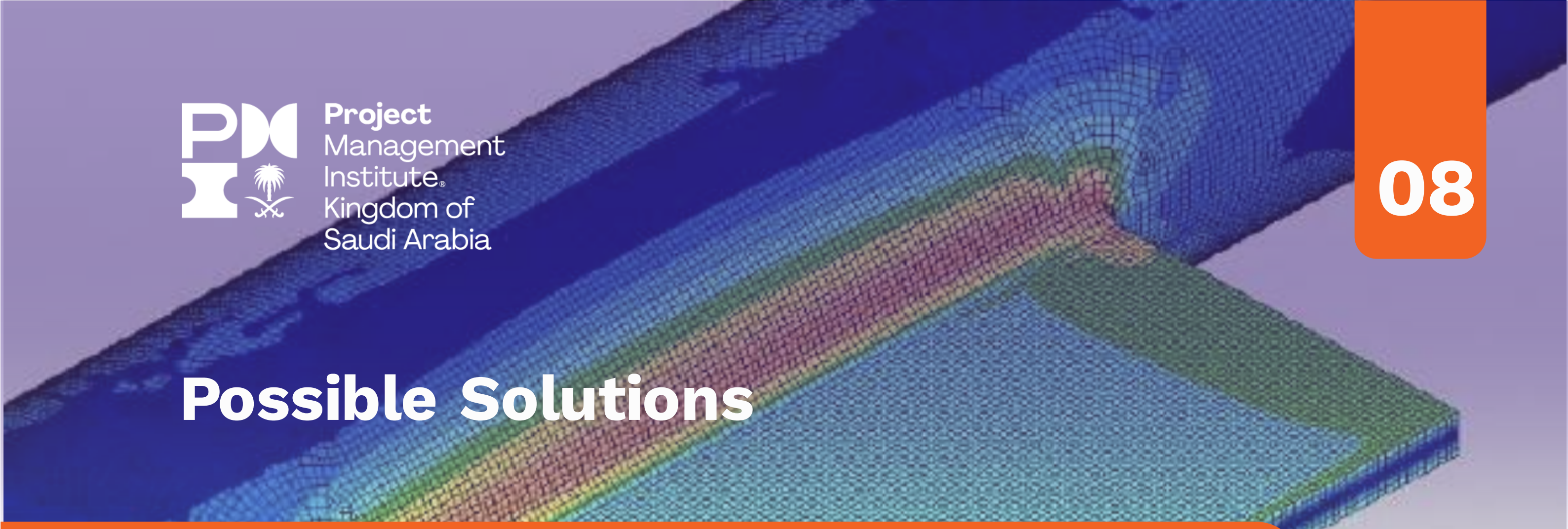
1. Increase the resonant velocity of the member.
2. Reduce the likelihood of vortices forming around the member.
3. Increase the damping of the beam to reduce the effect of vibration.

With each possible solution, there are advantages and disadvantages to the solution, which were evaluated considering the status of the existing structure.

Approach #1 Since the resonant velocity of a member is a function of the Strouhal number, the natural frequency of the member, and the depth of the member in the direction of fluid flow, there are four approaches to increase the resonant velocity:

- Increase the diameter of the pipe member.
- Decrease the unbraced length of the pipe with additional bracing.
- Change the end conditions from pinned or simply supported to fixed.
- Decrease the Strouhal number by changing the member profile to a shape member with sharp, flat sides.

Increasing the resonant velocity of the member is best suited for projects still in the design phase.This solution has a significant material impact to an existing structure and the largest of the three, since existing beams would need to be completely replaced, or additional materials or braces will need to be added to the structure.



Possible Solutions

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Approach #2 Reducing the likelihood of vortices is primarily achieved with helical strakes, fairing, or other flow disrupters. Flow disrupters are also best addressed during the design phase of a project, as they may have a large impact on wind area/loading. The issue with using flow disrupters on lattice structures is that they lose their effectiveness when fluid flow is not perpendicular to the axial direction of the structural member, or if interference effects leads to nonlaminar flow at the member.

Approach #3 Increasing the damping of the beam is achieved by increasing the Scruton number. This can be achieved with two approaches:

- Increase the mass of the system.
- Add an external damper.

Either option can implemented at any point during the project phase, since the overall weight and area impact to a steel structure can be minimized.

Project Constraints Driving Implemented Solution

Contrary to the normal design process, construction was the main driver for any remediation study. Many constructability methods were studied, and evaluation matrixes were performed before deciding on the optimum procedure. These also considered the criticality of this path on the overall schedule of the project in addition to the secondary factor of cost. or this project, the following constraints

- Structure had already been constructed at site, with assistance of rope access.
- A maximum of three months was the available window for performing the work before the site was expected to go live.
- Permission for welding activities or hot work was not allowed.
- Safety precautions needed to be fulfilled.
- The length and weight of any supplementary members to be added needed to be controlled for elevations of up to 200 m and would need to be easily assembled.

An additional constraint was the constructability method, since the mobile crawler crane used for construction was no longer available for use. Mobilizing another crawler crane would take many months for assembly and certification of a crane with a 200 m reachable boom. Additionally, the 250 x 60 m clear area for assembling such crane was no longer available.

Project Constraints Driving Implemented Solution

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Fig. 7 Crawler crane previously used for derrick installation.

Modifications using a tower crane braced against the derrick structure was therefore proposed. The disadvantages of this type of crane requires the installation of a 7 x 7 x 1.7 m deep foundation. Also consumed daily operation time would be high since the operator must climb up/down to/from the top side cabinet. Finally, the loads provided by the tower crane supplier, were larger than could be handled by the tower at design wind conditions.

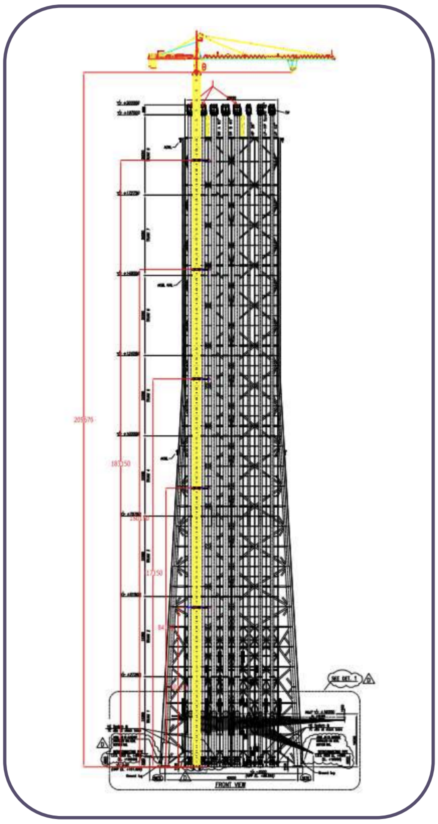


Fig. 8 Proposed tower crane configuration.

A scaffolding structure was also considered for access to working points along the structure. The design and installation of a scaffolding tower 190 m tall meant that the scaffolding was expected to exceed 6000T and 80,000 cubic meters and would require at least four months to construction and deconstruction.

It was therefore decided that the best course of action for the solution implementation would

Project Constraints Driving Implemented Solution

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

be through the rope access method, previously successfully used in the construction for the Ras Tanura derrick. This approach also had challenges.

Fig. 9 – Photo of rope access work

The weight of any installed solution would be limited to that which could be installed by a worker through rope access – approximately 150 kg. Sourcing rope access technicians was also difficult, as there were no local IRATA member companies. The importance of safety and with the impact of the COVID-19 pandemic also made it difficult to mobilize technicians from abroad. But, since rope access was the most optimum solution, the client gave a green light to start mobilizing the same rope access crew which utilized previously for the erection of the Ras Tanura derrick structure.

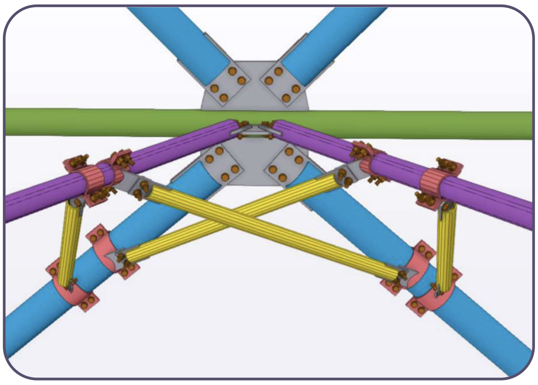


Fig. 10 Clamped Brace Solution Details

With this solution, four braces needed to be installed on each member of concern. Additionally, these braces were spanning between two members that were aligned on two different axes, making installation difficult. Finally, when the flexibility of the bracing points on the diagonal Saudi Aramco: Public members were considered, the resonant velocity was not increased enough to avoid vibration, and the added load from the braces could increase loading on the original member end plate.

The use of helical strakes was considered, but required an additional wind interference study, inside the structure, leading to doubts on the effectiveness of this solution. External tuned dampers were also considered, but the need to tune dampers to multiple geometries together with lead times for the dampers themselves,exceeded time constraints. Therefore, this approach was not chosen. The most practical option and the solution finally chosen was to add mass to the structure to reduce the vibration amplitude of members that were prone to vibration.

Chosen Solution Details

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

A clamped mass added to one half of the length of each beam in question was implemented.
This was calculated to functionally increase the Scruton number, reduce the displacement amplitude, and decrease the expected stress in the end connections, while not increasing the static load enough to cause weak axis bending issues in the existing gusset plates.
For this solution, the clamp size and stiffener size were standardized across the entire structure to minimize fabrication differences and construction lead times. The only variable parameters were the stiffener thickness for different diameter pipes, and then the length of the stiffener for different beam length.

Proposal	Description	Advantages	Disadvantages	Conclusion
Crawler Crane with Scaffolding	Minimum 1600 tons crawler crane with booster boom or 3200 tons CC Crane	-Proven, traditional lifting method -Highest weight capacity -Can reach high elevations -Hot work cut and welds are available	-No space for assembly -Very long time for mobilizing and assembling -Long time will be consumed for operation -Derrick foundation failed with scaffolding dead weight around 6000 tons	Not Viable
Tower Crane with Scaffolding or Rope Access	One tower crane supported with the derrick structure covering the whole derrick plan.	-Proven, traditional lifting method -High weight capacity -Can reach high -Hot work cut and welds are available	-3 months for mobilizing and assembly with big concrete foundation -Operation is not safe for operator to climbing up and down 220 meters by monkey ladders in daily with an assistant -Stress analyses connection with existing Derrek was failed -Derrick foundation failed with scaffolding dead weight around 6000 tons	Not Viable
Rope Access only	Approved IRATA certified technicians for working at heights easily with	-Easy mobilization -Easy setup, rigging and erection -Progressing is high	-Hot work cut and welds are not available -The safety precaution is high -Low weight limit (max 150kg) for easy handling by rope access crew	Viable
	approved tools and equipment	-All narrows spaces at any height are reachable -Scaffolding not required		

Table 1 – Construction Method Decision Matrix

The added mass solution proposed was to roll two plates around the existing pipe members. These plates could be standard grade steel. The stiffener was welded to clamp assemblies, and then installed in pairs (one on top and one on bottom) to the member to be stiffened. The clamp plates were installed with bolts, and a gap between the clamp plates allowed full clamping action on the pipe brace to ensure that no slippage would occur.

Chosen Solution Details

By Saad M. Qahtani, Abdelbadie Zahran, Gregg Williams, Miguel Ibáñez Mayayo .

Depending on the amount of weight needed, the clamped section was designed as shown below.

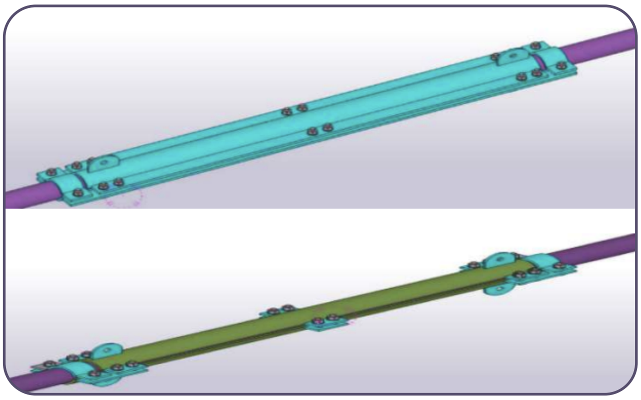


Fig. 11 3D View of the clamped added mass assembly

Literature

1. ASCE/SEI 7-05 (2005) Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers.

2. AIJ (1982). Stack Stuctural Design and Construction Code, Architectural Institute of Japan.

3. Blevins, R.D. Flow-induced vibration. Van Nostrand Reinhold 1990 2nd Edition.

4. CICIND (2010) Model Code for Steel Chimneys, International Comittee for Industrial Construction.

5. ANSI/AISC 360-16 (2019) Specification for Structural Steel Buildings, American Institute of Steel Construction.