Assessment of the Technical Condition of a Deteriorated Heat Pipeline Overpass before Repair

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Abstract. Pipeline overpasses are special structures common in many fields of industry. The magnitude and origin of damages to a heat pipeline overpass are presented and discussed. One of the more typical elements, a vertical truss with the smallest cross section, was chosen for an assessment of its resistance. The former design codes, valid at the time of the design, construction, and use of the overpass significantly differ from the current design codes. The calculations were performed with the use of three design codes, which were valid during the period of 60 years of use of the overpass. The results are summarized and compared.

1. Introduction
The application of pipeline overpasses in urban construction is generally limited to carrying heat pipelines over roads, rivers, and railways. However, pipeline overpasses are very common in many industrial fields. In factories they are used to join different industrial halls transporting raw materials, products, water or vapors [1], [2]. The pipelines are usually arranged on many kilometers of supporting structures above terrain, which makes them sensitive to corrosion. Pipeline overpasses are special industrial structures [3], and their safe use is under the conditions of proper construction and appropriate application of structural and material solutions.

The object of the study is a deteriorated heat pipeline overpass in a chemical plant. The magnitude and origin of damage to the steel and reinforced concrete elements of the overpass are presented and discussed. One structural element, the most typical compressed vertical truss of the smallest cross section, was chosen for an assessment of its resistance. The approach to structural design and the design codes has changed significantly in the period of 60 years of use of the overpass. Therefore, the calculations are performed with the use of three design codes valid during that period.

2. Description of the assessed overpass structure
The assessed overpass was constructed in the 1960’s. It is about 265 m long and it consists of supporting structures for heat pipelines supplying the plant and the city. The overpass is a space truss spanning 18.0, 21.0, and 24.0 m. The connectors are 6.0 m long. Pinned and fixed supports are applied. The pinned supports are two-dimensional reinforced concrete frames with two girders which are perpendicular to the longitudinal axis of the overpass. The fixed supports consist of rolled steel profiles forming space truss columns. The fixed supports are applied at the ends of every second segment approximately. The
rest of the supports at the ends of truss bridges are pinned ones. The height of the supports is about 7 m above the terrain level. On the supports, the steel space trusses are collocated with a slope of 3 ‰. They consist of two two-dimensional trusses with X shape bracings at the top and at the bottom (see figure 1). The top and bottom chord of the truss are joined by verticals and diagonals made of inverted “V” shape profiles. The truss is divided into variable sections between 4210 mm and 2280 mm. The steel profiles of different cross sections are used. Top and bottom chords are done with double “C” shape profiles: 240, 220, 200, 180 and 160 mm, spaced every 160 mm. The top and bottom chords lay in parallel planes with fixed external distance between profiles of 2930 mm. Therefore for different C-shape profiles used the distance between their axes varies from 2450 mm to 2610 mm. Change of the distances between axes occurs even twice within one segment of the bridge. Application of verticals and diagonals allowed for decreasing of gusset plates sizes, or even their elimination in case of joints of chords and verticals (see figure 2). For the verticals, angle profiles with unequal legs ∟60×40×6 and ∟75×50×8 were applied. For the diagonals the following profiles were used: double C-shapes 160 and angle profiles 2∟120×80×10, 2∟100×75×8, and ∟75×50×8. The ends, the top and bottom chords of the truss are supported on transverse girders 7.50 m long. The transverse beams consist of double C-shape profiles 180. On the transverse girders, also platform grates and balustrades are supported. The number of pipelines carried by the overpasses is usually 16, with a maximum of 23 pipelines (see figure 3).
3. Brief description of damage to overpass structure

Evaluation of the type and magnitude of damage to the overpass is based on a site inspection carried out from the terrain level and the platform level. The overpass is made of carbon steel and located on the premises of a chemical plant, which implies frequent renovation of the paint coating [2]. Additional factors accelerating the corrosion process might be leakages from the technological installations, and improperly designed compensation of the pipelines resulting in excessive displacements on the supports. The closed zones in the truss are particularly sensitive to corrosion, which do not allow for fast drainage of rainwater (see figure 4) and not the zones with inappropriate anticorrosive protection, for example the connections of the truss elements (see figure 5). A very high level of corrosion can be noted at the bottom parts of the overpass, especially the nodes. Those are point as well of surface corrosion. No pitting corrosion was observed. The corroded zones are not located in the most stressed regions of the structure, and the decrease of the structural resistance due to corrosion is not significant.

Figure 3. Overpass cross section.
The structure of the overpass was designed according to the design codes valid in the 1960’s. During many years of use, the loading was changed. New pipelines were added and some of the verticals and diagonals of the truss had to be strengthened (see figure 6). Some verticals and bracings of the truss were deformed, probably during the assembly of the additional pipelines.

Reinforced concrete frames were not additionally protected against the influence of the aggressive environment. They were subjected to cyclic environment actions like: water saturation and drying, freezing, and thawing. It led to deterioration of the external surface of the concrete. Cracks and micro cracks allowed for deeper penetration of water and air, which resulted in corrosion of steel reinforcement. Changes to the volume of the corroded steel reinforcement were followed by splitting and spalling of concrete and as a consequence local losses of concrete cover (see figure 7 and figure 8). The concrete layer up to 4 cm lost its ability to protect the steel reinforcement. Depth of the carbonization was evaluated based on the phenolphthalein method.

**Figure 4.** Corrosive damage in the vicinity of the closed zone of the truss.

**Figure 5.** Example of corrosive damages in the truss node.

**Figure 6.** Strengthening of a vertical truss with a steel plate.

**Figure 7.** Damage to a column of the reinforced concrete frame.
4. Preliminary study for the compressed vertical truss
Introducing changes and strengthening of the structure, which was designed with the use of design codes PN-B, it is necessary to prove it fulfills also the requirements of the current codes [4]. The structure was designed according to the Code PN-B-03200:1962 [5]. Its strengthening was designed most likely with the design codes PN-B-03200:1976 [6], PN-B-03200:1980 [7], or PN-B-3200:1990 [8]. During that time period of use also the rules for estimation of effects of actions have been changed. The previous codes PN-B differ significantly from the Eurocodes, [9], [10], [11]. Therefore, three design codes are used for a comparative calculation of the resistances. For the analysis, the most typical vertical truss of the smallest cross section is selected. It is a two angle profile (see figure 9) joined with lacements every $l_1 = 470$ mm.

Figure 8. Damage to frame girders – loss of concrete cover and corrosion to reinforcement steel.

Figure 9. Cross section of the two angle profiles of the vertical truss.
Geometrical characteristics of the cross section:
\[ A = F = 2 \times 5.68 = 11.36 \text{ cm}^2, \]
\[ J_y = 2 \times (7.12 + 5.68 \times 6.99^2) = 569.3 \text{ cm}^4, \]
\[ i_y = 1.88 \text{ cm}, \quad i_z = 7.08 \text{ cm}, \quad \text{for a single arm: } J_\eta = 4.15 \text{ cm}^4, \quad i_\eta = 0.86 \text{ cm}. \]

In the calculation below, the original denomination from the corresponding codes is used.

### 4.1. Maximum value of the axial force \( P \) in the vertical truss according to PN-B-03200:1962

Assuming that the truss chords consist of C-shape profiles 180, the in-plane buckling length equals:
\[ l_{wy} = 0.8 \times (293.0 - 18.0) = 220.0 \text{ cm}, \]
and in the plane perpendicular to the plane of the truss:
\[ l_{wz} = 293.0 - 18.0 = 275.0 \text{ cm}. \]

Therefore, correspondingly the slenderness of the bar is equal to:
\[ \lambda_y = 220.0/1.88 = 117.0 \quad \text{and} \quad \lambda_z = 275.0/7.08 = 38.8. \]

The maximum characteristic force should fulfill the condition:
\[
P \leq k \times \beta \times F \quad (1)
\]

where:
- \( k \) – permissible steel compressive stress, assumed 150 MPa
- \( \beta \) – buckling coefficient corresponding to the highest slenderness

In the plane of the truss, the buckling coefficient for steel St3 equals \( \beta = 0.438 \). The maximum force in the vertical truss calculated from equation (1) at in-plane buckling should be smaller than:
\[ P \leq 15.0 \times 0.438 \times 11.36 = 74.63 \text{ kN}. \]

The slenderness of the single arm equals:
\[ \lambda_1 = l_1/i_1 = 47.0/0.86 = 54.7. \]

Therefore, the equivalent slenderness equals:
\[
\lambda^*_e = \lambda_z \sqrt{1 + \left(\frac{\lambda_y}{\lambda_z}\right)^2} = 38.8 \sqrt{1 + \left(\frac{54.7}{38.8}\right)^2} = 67.1,
\]
and the buckling coefficient \( \beta = 0.757 \).

The maximum force in the vertical truss calculated from equation (1) at buckling in the plane perpendicular to the truss plane should be smaller than:
\[ P \leq 15.0 \times 0.757 \times 11.36 = 129.00 \text{ kN}. \]

### 4.2. Resistance of the vertical truss according to PN-B-03200:1990

Single profiles fulfill the requirements established for cross section class 3, and for the compound cross sections class 3 was assumed. The resistance of the cross section at axial compression equals:
\[ N_{rc} = A \times f_d = 11.36 \times 21.5 = 244.24 \text{ kN}. \]

Design axial force \( N \) in the compressed vertical, considering general stability (buckling coefficient \( \varphi \)), should not be higher then
\[
N \leq \varphi \times N_{rc}. \quad (2)
\]

The in-plane buckling length of the vertical equals \( l_e = l_w = 220.0 \text{ cm}, \) and the out of plane buckling length \( l_\eta = 275.0 \text{ cm}. \)

The slenderness of the bar equals correspondingly:
\[ \lambda_y = 220.0/1.88 = 117.0 \quad \text{and} \quad \lambda_z = 275.0/7.08 = 38.8. \]
The non-dimensional slenderness at the in-plane buckling:
\[ \bar{\lambda}_y = \frac{\lambda_y}{84\sqrt{215/215}} = 1.39. \]

The coefficient of general instability from the table 11 of the code PN-B-03200:1990 (curve c):
\[ \varphi = 0.379. \]

Design axial force \( N \) in the compressed vertical truss, at in-plane buckling, calculated from equation (2), should not exceed:
\[ N \leq 0.379 \times 244.24 = 92.57 \text{ kN}. \]

At buckling in the plane perpendicular to the plane of the truss, the equivalent slenderness of the two armed vertical (\( m = 2 \)) equals:
\[ \lambda_m = \sqrt{\frac{\lambda_y^2 + \frac{m}{2} \lambda_h^2}{38.8^2 + \frac{2}{54.7^2}}} = 67.1 > \lambda_z. \]

The design resistance of the cross section at axial compression was calculated as for elements of class 4 according to the equation:
\[ N_{Rc} = \psi \times A \times f_d \quad (3) \]

For calculation \( \psi = \varphi \) was assumed. The slenderness of a single arm equals: \( \lambda_1 = 54.7. \)

The non-dimensional slenderness at the out-of-plane buckling:
\[ \bar{\lambda}_m = \frac{54.7}{84\sqrt{215/215}} = 0.65. \]

The coefficient of general instability from the table 11 of the code (curve c): \( \varphi = 0.776. \)

The resistance of the cross section at axial compression equals:
\[ N_{Rc} = \psi \times A \times f_d = 0.776 \times 11.36 \times 21.5 = 189.5 \text{ kN}. \]

The non-dimensional slenderness of the vertical at flexural buckling in the plane perpendicular to the plane of the truss was calculated from the equation:
\[ \bar{\lambda}_{mz} = \frac{\lambda_m \sqrt{\varphi}}{\lambda_p} = \frac{67.1}{84.0} \sqrt{0.776} = 0.70. \]

The coefficient of general instability from table 11 of the code PN-B-03200:1990 (curve b) \( \varphi = 0.841. \)

The design axial force \( N \) at buckling in the plane perpendicular to the plane of the truss, calculated from equation (2), should not exceed:
\[ N \leq 0.841 \times 189.5 = 159.37 \text{ kN}. \]

4.3. Check of the ultimate limit state according to PN-EN 1993-1-1:2006

The resistance of the compound cross section is calculated in an approximated manner, as for single profile [12]. The detailed calculations including initial imperfections, given in Eurocode 3, were omitted. Assuming that for the scope of comparisons, the approximated calculation of the resistance of the cross section is sufficient. Buckling lengths of the vertical truss in and out of the plane of the truss are: \( L_{cr,y} = L_{cr,z} = 275.0 \text{ cm}. \)

Slenderness of the element:
\[ \lambda_y = \frac{L_{cr,y}}{l_y} = \frac{275.0}{1.88} = 146.3. \]

The non-dimensional slenderness of the bar at buckling in the plane of the truss equals:
\[ \bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{146.3}{93.9} = 1.56. \]
According to the code [11], the cross section considered corresponds to the buckling curve “b” with imperfection parameter $\alpha = 0.34$.

Parameter of the instability curve $\Phi_y$:
$\Phi_y = 0.5[1 + \alpha(\tilde{\lambda}_y - 0.2) + \tilde{\lambda}_y^2] = 0.5[1 + 0.34(1.56 - 0.2) + 1.56^2] = 1.95$

The reduction factor for buckling $\chi_y$:
$$\chi_y = \frac{1}{\sqrt[1.95^2-1.56^2]} = 0.32.$$

Design buckling resistance about axis $y$:
$$N_{b,Rd} = \frac{X_y A f_y}{\gamma_{M1}} = \frac{0.32 \times 11.36 \times 23.5}{1.00} = 85.43 \text{kN}.$$  
Design axial force at in-plane buckling should fulfill the condition: $N_{Ed} \leq N_{b,Rd} = 85.43 \text{kN}.$

At out of plane buckling, the slenderness equals:
$$\lambda_x = \frac{L_{cr,x}}{i_x} = \frac{275.0}{7.08} = 38.84.$$  
The non-dimensional slenderness of the bar at out of plane buckling equals:
$$\tilde{\lambda}_x = \frac{38.84}{93.9} = 0.41.$$  

Parameter of the instability curve $\Phi_x$:
$\Phi_x = 0.5[1 + \alpha(\tilde{\lambda}_x - 0.2) + \tilde{\lambda}_x^2] = 0.5[1 + 0.34(0.41 - 0.2) + 0.41^2] = 0.62$

The reduction factor for buckling $\chi_x$:
$$\chi_x = \frac{1}{\sqrt[0.62^2-0.41^2]} = 0.92.$$  
The buckling resistance of the element, about axis $z$:
$$N_{b,Rd} = \frac{X_x A f_x}{\gamma_{M1}} = \frac{0.92 \times 11.36 \times 23.5}{1.00} = 245.60 \text{kN}.$$  
The design axial force at out of plane buckling should fulfill the condition: $N_{Ed} \leq N_{b,Rd} = 245.60 \text{kN}.$

4.4. Check of a single arm of the profile according to PN-EN 1993-1-1:2006
For a single angle profile buckling length equals $L_{cr,v} = 47.0 \text{ cm}$. Therefore, the slenderness of element equals: $47.0/0.86 = 54.7$. At out-of-plane buckling, the slenderness equals:
$$\tilde{\lambda}_v = \frac{L_{cr,v}}{i_z} = \frac{47.0}{0.86} = 54.70.$$  
The non-dimensional slenderness:
$$\tilde{\lambda}_v = \frac{54.70}{93.9} = 0.58.$$  
The non-dimensional slenderness of the angle profile about axis $v$:
$$\tilde{\lambda}_{eff,v} = 0.35 + 0.7 \cdot \tilde{\lambda}_v = 0.35 + 0.7 \cdot 0.58 = 0.76.$$  
According to the code [11], the cross section considered corresponds to the buckling curve “b” with imperfection parameter $\alpha = 0.34$. 


The parameter $\Phi_v$:

$$\Phi_v = 0.5[1 + \alpha(\tilde{\lambda}_{\text{eff},v} - 0.2) + \tilde{\lambda}_{\text{eff},v}^2] = 0.5[1 + 0.34(0.76 - 0.2) + 0.76^2] = 0.88$$

The reduction factor for buckling $\chi$:

$$\chi_v = \frac{1}{\Phi_v + \sqrt{\Phi_v^2 - \lambda_{\text{eff},v}^2}} = \frac{1}{0.88 + \sqrt{0.88^2 - 0.76^2}} = 0.76.$$  

The buckling resistance about axis $z$:

$$N_{\text{ch,Rd}} = \frac{\chi_v \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.76 \cdot 5.68 \cdot 23.5}{1.00} = 101.44 \text{kN}$$

The design axial force at out of plane buckling should fulfill the condition:

$$N_{\text{Ed}} \leq 2 \times N_{\text{ch,Rd}} = 2 \times 101.44 = 202.88 \text{kN}.$$  

4.5. Comparison of the results

The results of the calculations of the resistance for the selected vertical truss are summarized in Table 1 below. The comparison was made for the chosen three design codes valid over the period of 60 years of the use of the overpass. The former design codes valid at the times of the design, construction and use of the overpass significantly differ from the current design codes. Therefore, the assessment of the actual technical condition of the structure, for lack of the technical documentation, is problematic [13]. Information about the condition of the structure, and possibility of its further use and development may be achieved by control calculations according to the current design codes. It has to be underlined that in the code PN-B-03200:1962, the cross sections were design using the permissible stress design method. Permissible stress $k = 150$ MPa is defined as a ratio of yield strength $R_e = 240$ MPa to the safety coefficient $n_e = 1.6$. The maximum axial compressive force in the cross section is calculated with consideration of the effect of buckling effect, and it is related to the characteristic loads on the overpass. For the calculation of the resistance of the vertical truss according to the codes PN-B-03200:1990 and PN-EN 1993-1-1:2006 the ultimate state method is used. The maximum design effect of actions for the vertical truss is defined by application of partial safety factors: material factors, partial factors for actions, and factors from a combination of actions. Effects of actions assumed according to the Eurocodes are usually higher than those obtained from the codes PN-B, especially environmental actions. The highest are the design effects of permanent actions, transferred from the pipelines to the overpass structure: self-weight of the pipelines, thermal insulation, water filling the pipelines, weight of the platforms, [9] and [10]. It should be mentioned that the safety factor for permanent loads according to PN-B equals $\gamma_{G,PN} = 1.10$, and according to the Eurocodes $\gamma_{G,EC} = 1.35$. Taking into account the above-mentioned comments, the design resistance of the vertical according to the code [8] is highest and according to the Eurocode [11] is lowest. The significant difference in estimation of the resistance of the structural element might result that additional pipelines were added without strengthening of the structure.

| Code                                | Max. characteristic loading acc. to PN-B-03200:1962 | Max. design effect of actions |
|-------------------------------------|-----------------------------------------------------|-------------------------------|
|                                     | $P$ [kN]                                           | $N$ [kN]                      | $N_{\text{Ed}}$ [kN]          |
| In-plane of the truss               | 74.63                                              | 92.57                         | 85.43                         |
| Out-of-plane of the truss           | 129.00                                             | 159.37                        | 245.60                        | 202.88                         |
5. Summary and conclusions

A heat pipeline overpass in a chemical plant was inspected to identify the damage to the structure. The structure was found to be sensitive to corrosion due to insufficient paint coating, leakages from the technological installations and excessive displacements on the supports resulting from lack of the compensation of the pipelines. The corroded zones were identified in the steel structure and reinforced concrete structure. During 60 years of use the loading to the overpass had been increasing by addition of new pipelines, which was not always accompanied by adequate strengthening.

For assessing the resistance, the most typical vertical truss of the smallest cross section was chosen, with double angle profile 60x40x6 joined with lacings. The maximum axial compressive force in the cross section was calculated with consideration of the effect of the buckling effect, and it is related to the characteristic loads on the overpass. The calculations are performed with the use of three design codes valid during the period of design and use of the overpass: PN-B-03200:1962, PN-B-03200:1990, and PN-EN 1993-1-1:2006. The codes significantly differ in the approach to the structural design. While the oldest codes were based on the permissible stress design method, the newer codes are basing on the limit state design. Also the loadings to the structure, especially the environmental actions, have been changed. The design resistance of the truss vertical calculated according to the Eurocodes was found to be the lowest. For modernization, the structure should be recalculated and strengthened to fulfill the requirements of the Eurocodes.

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