

Integrated Analytical and Finite Element Assessment of Stress and On-Bottom Stability for a Deepwater Offshore Pipeline in the Gulf of Guinea

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ABSTRACT

Offshore subsea pipelines in the Gulf of Guinea are subjected to the combined effects of deepwater hydrostatic pressure, soft-clay seabed conditions and a high-intensity metocean environment. In this paper, an integrated analytical-numerical approach is developed for evaluating the stress and on-bottom stability of one specific case: a 508 mm outer diameter, 15.88 mm wall thickness API 5L X65 pipeline in a 30 m free span at a water depth of 1000 m. Analytical formulations consistent with DNV-OS-F101, DNV-RP-F109, ASME B31.3 and ASME B31.4 (with DNV-RP-C203 used for fatigue) were coupled with a three-dimensional finite element model built in ANSYS Workbench and subjected to 100-year return period hydrodynamic loading derived from the Morison equation framework and applied as static and quasi-static loads. The finite element model was discretized with a hexahedral mesh of 49,841 nodes and 24,786 elements and verified for mesh independence. The model converged to a peak von Mises stress of 278 MPa at the mid-span bottom fiber, corresponding to an ASME B31.3 utilization ratio of 0.927 (278 MPa against a 300 MPa allowable), an ASME B31.4 combined-stress utilization of 0.686 and 61.8% of the specified yield strength. On-bottom stability was checked with a safety factor of 2.05 versus the minimum of 1.1 stipulated by DNV-RP-F109. A screening fatigue check using Miner's rule gave a cumulative damage ratio of 0.67 for a design life of 25 years, before application of a design fatigue factor and excluding vortex-induced vibration. The finite element and closed-form analytical stress results agreed to within 5%. In the sensitivity study, in which only the current velocity was varied, a 72% increase in velocity (0.29 to 0.50 m/s) produced an approximately 198% increase in drag force, and the buckling analysis indicated a need for engineered lateral restraint and sufficient burial depth at sites susceptible to lateral and upheaval buckling. For this configuration the 300 MPa allowable stress was reached at a diameter-to-thickness ratio of about 36, so the design value of 32 lies inside the safe window; this is a case-specific result for the stated span length, loading and support conditions and is not a general limit for other diameters, steel grades, spans or environments. The results provide validated stress, stability and screening-level fatigue benchmarks for this configuration and a workflow that can be recalibrated with site-specific data.

Keywords: Offshore pipeline, Finite element analysis, ANSYS, Gulf of Guinea, On-bottom stability, Von Mises stress, DNV-OS-F101, Fatigue damage, Lateral buckling

INTRODUCTION

Background

Offshore pipelines are the main method for moving hydrocarbons from subsea wellheads and risers to processing and export facilities. During their service life they are subjected to a complex combination of internal pressure, external hydrostatic pressure, thermal expansion, hydrodynamic loading due to waves and currents and pipe-soil interaction. As field developments move into deeper and more hostile environments, the structural integrity and on-bottom stability of these systems have become increasingly critical design considerations (Ujjwai & Prateek, 2020; Widiyanto et al., 2022).

Pipe stress analysis assesses the behavior of a piping system under the loads experienced during operation. It is generally categorized into five types: hoop stress, which is due to internal or external pressure and determines the wall thickness; axial stress, caused when the pipe is prevented from expanding or contracting due to thermal or pressure loading; bending stress, resulting from concentrated forces or imposed displacement; torsional stress, arising from body forces that create rotational moments; and fatigue stress, which accumulates from recurrent cyclic loading (Ujjwai & Prateek, 2020; Smith & Patel, 2025). Concurrently, stability analysis looks at the pipeline's ability to stay in place and maintain its structural shape on the seabed under wave, current and soil-movement loading; insufficient stability can result in lateral displacement, free-span formation, upheaval buckling or loss of structural integrity.

Offshore pipelines in the Gulf of Guinea have long suffered from stability problems, due to the difficult seabed and hydrodynamic conditions of the region. Free spans are formed when the erosion of the soft-clay seabed results in the removal of support for the pipeline sections, exposing them to bending and fatigue loading not accounted for in the original design. Such spans, if not addressed, can increase in size under the influence of continuous scour and current action, increasing the likelihood of excessive deflection, local overstress or rupture (Ekwere et al., 2021). Small variations in wall thickness, coating condition or soil strength can shift a pipeline segment from a state of global stability into local instability (Adumene et al., 2021), so the region continues to require a dedicated, quantitative stability analysis rather than a generic, imported one. This confluence of structural, economic and environmental risk motivates a stability-analysis methodology developed specifically around Gulf of Guinea conditions rather than adapted from other offshore provinces (Widiyanto et al., 2022).

Aim and Objectives

This study aims to perform an integrated stress and stability analysis of an offshore pipeline in the Gulf of Guinea using finite element analysis cross-validated with closed-form analytical methods. The results are demonstrated for one configuration, a 508 mm outer diameter, 15.88 mm wall thickness API 5L X65 pipeline in a 30 m free span at 1000 m water depth; the extent to which they can be transferred to other configurations is stated explicitly in Sections 4.3 and 4.6. The specific objectives are: (i) to develop a three-dimensional finite element model of an API 5L X65 pipeline in ANSYS Workbench for the offshore environment of the Gulf of Guinea, (ii) to subject the model to 100-year return period design loads derived from Gulf of Guinea metocean data, applied as static and quasi-static loads, for stress and stability analysis, (iii) to check the resulting stress and stability outputs against the von Mises stress criterion, DNV-OS-F101, DNV-RP-F109, ASME B31.3 and ASME B31.4 (the result for each is reported in Section 4.5), and (iv) to validate the accuracy of the numerically obtained strength and stability results against independent empirical and analytical methods.

LITERATURE REVIEW

Stress and Stability Analysis of Offshore Pipeline

The stress and stability of offshore pipelines have been extensively studied either by analytical design-code checks or finite element simulation in isolation. Anas, Alam and Shariq (2023) review failure mechanisms under combined bending and pressure, while Arumugam et al. (2023) synthesise the influence of axial compressive stress and internal pressure on pipeline networks; both highlight the sensitivity of stress response to geometric and loading parameters but stop short of coupling the analytical checks to a validated finite element model. Bransby, Low and Zhu (2022) and Yu, Hu and Kong (2025) look at pipe-soil interaction in a great deal of detail, giving useful constitutive knowledge for burial and lateral movement, but usually in generic or non-Gulf of Guinea soil and metocean conditions. In offshore pipeline integrity assessment, Adumene et al. (2021) specifically consider material and parametric uncertainty, stressing the need for locally calibrated input data instead of generic default values.

Environmental and Geotechnical Considerations in the Gulf of Guinea

The subsea environment in the Gulf of Guinea is very challenging, where currents, waves and a soft cohesive seabed combine to impose hoop stress and hydrodynamic drag and lift loads on pipelines, which are strongly coupled in ways that are not easily represented by simple single-parameter metrics (Ekwere et al. 2021; Ojie et

al. 2026). Repeated exposure and burial in strong current regimes along the pipeline route lead to uneven support and measurable effects on the stability of suspended sections during lateral excursions (Wang, Zhang & Liu, 2023). Corrosion rates calibrated for regional conditions may also be significantly different from generic assumptions and reduce the wall-thickness safety margin used by stability calculations over the asset's life (Roberts & Johnson, 2023). Such effects have economic and environmental implications. Unexpected repair, inspection and downtime costs increase quickly once stability problems arise (Roberts & Kumar, 2024). Any discharge of hydrocarbons in the sensitive marine and coastal environment of the Gulf of Guinea can impact ecology, fisheries and coastal communities (Adumene et al., 2022).

Gap in Research

Comparatively missing in the literature is a single cross-validated workflow that runs a given pipeline configuration through geometry, meshing, hydrodynamic loading, code-based stress and stability checks, buckling analysis and fatigue assessment using Gulf of Guinea-specific metocean and geotechnical data throughout. This study directly addresses this gap by using the finite element model as an independent stress-and-stability predictor and as a cross-validation benchmark against the closed-form analytical methods embedded in DNV-OS-F101, DNV-RP-F109, ASME B31.3 and ASME B31.4.

Significance of the Study

Beyond its immediate technical outputs, the aim of this study is to offer practitioners and regulators involved in Gulf of Guinea pipeline developments a validated and replicable analytical-numerical workflow, rather than a single case-specific result. The methodology provides a level of assurance that neither a purely analytical nor a purely numerical approach can provide alone by explicitly cross-validating the ANSYS finite element predictions against closed-form DNV and ASME formulations for the stress and stability results. The quantitative benchmarks established here (peak von Mises stress, on-bottom stability safety factor, cumulative fatigue damage ratio and code utilisation ratios) are intended to function as reference values for preliminary integrity screening of the analysed configuration. For other configurations they indicate the expected order of magnitude only and must be re-derived or recalibrated within the validity ranges stated in Section 4.3, and project-specific analysis remains necessary.

MATERIALS AND METHODS

The methodology combines code-compliant analytical modelling with finite element analysis to provide a validated, industry-standard assessment. The pipeline configuration, material properties and environmental loads are region-specific to the Gulf of Guinea (Ekwere et al., 2021). The inputs describe a single representative configuration (Tables 1 and 2), and the results strictly apply to the range of conditions that those tables define.

Pipeline Geometry, Material Properties and Governing Sectional Equations

Table 1 and Table 2 summarise the geometric and material input parameters used to define the pipeline model. The pipe cross-sectional area is first estimated using the thin-wall approximation, valid when $t \ll D_m$, and then verified using the exact annular expression:

$$A \approx \pi \cdot D D_m \cdot t \quad (1)$$

$$AA_{exact} = \frac{\pi}{4} \cdot (D^{o2} - D^{i2}) \quad (2)$$

The second moment of area and elastic section modulus, required for the bending-stress and buckling checks in Sections 3.7 and 3.9, are then computed as:

$$I = \frac{\pi}{4} \cdot (r^{o4} - r^{i4}) \quad (3)$$

$$Z = \frac{I}{r_o} \quad (4)$$

The steel volume and mass, used to verify the mass properties assigned in the finite element model, follow directly from the exact cross-sectional area:

$$V = AA_{exact} \times L, \quad m = \rho \rho_{steel} \times V \quad (5)$$

where A = cross-sectional area (m^2), D_m = mean diameter (m), t = nominal wall thickness (m), D_o/D_i = outer/internal diameter (m), I = second moment of area (m^4), Z = elastic section modulus (m^3), r_o/r_i = outer/internal radius (m), L = pipeline length (m), and ρ_{steel} = steel density (kg/m^3). Full symbol definitions are given in the Nomenclature.

Parameter	Value	Unit
Outer diameter, D_o	0.508	m
Internal diameter, D_i	0.4762	m
Nominal wall thickness, t_{nom}	0.01588	m
Corrosion coating thickness	0.0065	m
Concrete coating thickness	0.100	m
Effective outer diameter, D_{eff}	0.708	m
Free-span analysis length, L	30	m
Water depth, h	1000	m

Table 1. Pipeline geometry and configuration (source: project specification; Ekwere et al., 2021).

Parameter	Value	Unit
Young's modulus, E	2.07×10^{11}	Pa
Poisson's ratio, ν	0.30	—
Yield strength, σ_y	450×10^6	Pa
Density of steel, ρ_{steel}	7850	kg/m^3
Density of concrete coating, ρ_{conc}	3000	kg/m^3
Density of corrosion coating, ρ_{corr}	1300	kg/m^3
Coefficient of thermal expansion, α	1.16×10^{-5}	$^{\circ}C^{-1}$
Internal pressure, P_i	12	MPa
External pressure, P_o	10.06	MPa
Temperature differential, ΔT	21	$^{\circ}C$

Current velocity, V_c (100-yr)	0.29	m/s
Significant wave height, H_s (100-yr)	3.45	m
Peak wave period, T_p (100-yr)	17	s
Submerged weight, W_s	3663.14	N/m
Soil friction coefficient, μ	0.6	—
Seawater density, ρ_{sw}	1025	kg/m ³
Drag coefficient, C_D	1.05	—
Lift coefficient, C_L	0.9	—
Inertia coefficient, C_M	3.29	—
Wave–current heading angle, β	0	°
Near-bed wave orbital velocity, U_w (100-yr)	1.2×10^{-6}	m/s
Design flow velocity, V_{design} (Eq. 10)	0.29	m/s
Undrained shear strength of soft clay, s_u	5	kPa
Submerged unit weight of seabed soil	6	kN/m ³
Pipe–soil spring stiffness (vertical / lateral / axial)	$4.4 \times 10^3 / 3.8 \times 10^3 / 4.4 \times 10^2$	kN/m per m

Table 2. API 5L X65 material properties, design loading, hydrodynamic and geotechnical input parameters (source: API 5L; Ekwere et al., 2021). Hydrodynamic coefficients follow DNV-RP-F109 and pipe–soil stiffness follows DNV-RP-F105; the soil strength and unit weight are assumed values representative of soft deepwater clay, and the heading angle assumes collinear wave and current.

The hydrodynamic coefficients C_D , C_L and C_M and the wave–current combination follow DNV-RP-F109 for a current-dominated flow. The near-bed orbital velocity is negligible ($U_w \approx 1.2 \times 10^{-6}$ m/s), giving a Keulegan–Carpenter number $KC = U_w T_p / D_{eff} \approx 3 \times 10^{-5}$ and a current-to-wave velocity ratio V_c / U_w of order 10^5 , so the steady-current coefficients govern and the wave-induced inertia contribution is negligible. At $V_c = 0.29$ m/s and $D_{eff} = 0.708$ m the Reynolds number is of order 10^5 , and $C_D = 1.05$ and $C_L = 0.9$ were adopted as representative values for a rough, concrete-coated pipe lying on the seabed, with $C_M = 3.29$ for a pipe in contact with the seabed. These values are retained for a pipe with little embedment in soft clay; embedment would reduce the lift and drag, so the choice is conservative. The 100-year current velocity, significant wave height and peak period are taken from the regional data compilation of Ekwere et al. (2021), which is a project data set rather than a published standard; the current is assumed to act normal to the pipeline and collinear with the waves ($\beta = 0^\circ$), and the 100-year value was applied at pipe level without any boundary-layer reduction to the seabed, which is conservative. The geotechnical inputs represent a typical soft clay rather than measured values along a specific route, with an undrained shear strength $s_u = 5$ kPa adopted as representative of the upper seabed sediments that govern pipe embedment, a range of 2–10 kPa considered for soft, normally consolidated deepwater clays, and a submerged unit weight of 6 kN/m³ (Table 2). The seabed friction coefficient ($\mu = 0.6$) is a representative value; it controls the on-bottom stability factor and the lateral buckling wavelength (Eq. 27), so the stability and buckling results are conditional on it (Section 4.6).

Finite Element Model Development

A 3D model of the 30 m free span pipeline was developed in ANSYS Workbench 2025 R2 including the composite wall structure (steel, corrosion coating and concrete coating). The API 5L X65 steel was modeled by using a bilinear kinematic hardening (BKIN) material model to describe the post-yield behavior, and the mass properties in Table 2 were checked against Equation (5) before loading (volume 0.718 m³, mass 5,634.3 kg).

Mesh Generation and Convergence

The pipe wall was discretized using a structural hexahedral mesh with 24 elements around the circumference, with local refinement (element size 0.025-0.064 m) at the mid-span and support regions to capture stress concentrations. The mesh was predominantly hexahedral, converging to 49,841 nodes and 24,786 elements, with a bounding-box diagonal of 30.009 m and a minimum geometric edge length of 0.748 m (a property of the geometry rather than of the refined mesh). Mesh independence was confirmed using a sequential grid-refinement study, in which the relative change in peak von Mises stress between successive mesh densities was required to fall within an acceptable discretisation tolerance:

$$\varepsilon \varepsilon_{mesh} = |\sigma \sigma_n - \sigma \sigma_{n-1}| / \sigma \sigma_n \times 100\% \leq 2\% \quad (6)$$

where σ_n and σ_{n-1} are the peak von Mises stresses obtained on the current and previous mesh densities, respectively. The adopted mesh satisfied this criterion and the nonlinear solution met the Newton-Raphson force and displacement convergence criteria, confirming the numerical stability of the static solution.

Boundary Conditions and Loading

The pipe-soil interaction was modeled by nonlinear spring elements according to Beam-on-Elastic-Foundation theory, with stiffness and friction limits representing a soft clay ($s_u = 5$ kPa, with 2–10 kPa considered), with vertical, lateral and axial stiffnesses of 4.4×10^3 , 3.8×10^3 and 4.4×10^2 kN/m per m respectively (Table 2). The stiffnesses were derived from a shear modulus $G = 500s_u$ and $\nu = 0.5$ using the DNV-RP-F105 dynamic-stiffness expressions, with the axial value taken as the friction limit μW_s mobilised over 5 mm; all are assumed representative values for soft clay, not route-specific measurements. Fixed supports were adopted at both ends ($X = 0$ m and $X = 30$ m) to model a fully restrained configuration typical of welded connections to risers or flanges. Real span shoulders are more compliant than a fixed support, so this idealisation overestimates end restraint and underestimates the mid-span bending stress; the reported mid-span stress should therefore be read as a best estimate with respect to support stiffness and not as a conservative bound (Section 4.6). The loads applied were an internal pressure of 12 MPa, an external pressure of 10.06 MPa, a drag force of 12,375 N at mid-span, and a lift force of 8,250 N at mid-span. Static structural analysis, linear eigenvalue buckling analysis and nonlinear (Riks arc-length) buckling analysis were performed to capture, respectively, operational steady-state response, theoretical lower-bound buckling loads, and realistic collapse behaviour incorporating geometric imperfections and material nonlinearity.

Hydrodynamic Load Modelling

Hydrodynamic loads were computed using the Morison equation framework, which decomposes the total force into drag, lift and inertia components:

$$FF_D = \frac{1}{2} \cdot \rho \rho_{sw} \cdot CC_D \cdot DD_o \cdot V^2 \quad (7)$$

$$FF_L = \frac{1}{2} \cdot \rho \rho_{sw} \cdot CC_L \cdot DD_o \cdot V^2 \quad (8)$$

$$FF_I = \rho \rho_{sw} \cdot CC_M \cdot (\pi \cdot D^{o2} / 4) \cdot a \quad (9)$$

where F_D , F_L and F_I are the drag, lift and inertia forces per unit length (N/m), ρ_{sw} is seawater density (kg/m³), C_D , C_L and C_M are the drag, lift and inertia coefficients, D_o is the effective outer diameter (m), V is the design

flow velocity (m/s) and a is the near-bed flow acceleration (m/s^2). Current and wave-induced velocities were combined by vectorial superposition per DNV-RP-F109:

$$VV_{design} = VV_c + UU_w \cdot \cos(\beta) \quad (10)$$

with the near-bed wave orbital velocity U_w obtained from linear wave theory using the significant wave height H_s , peak period T_p , wave number k and water depth h . At $h = 1000$ m the sea state ($H_s = 3.45$ m, $T_p = 17$ s) is in deep water, where the linear-theory near-bed orbital velocity decays with $1/\sinh(kh)$ and is expected to be negligible compared with the current. U_w was evaluated from linear theory as $U_w = \pi H_s / [T_p \cdot \sinh(kh)]$, with the wave number k from the dispersion relation; for $H_s = 3.45$ m, $T_p = 17$ s and $h = 1000$ m, $kh \approx 13.9$ and $U_w \approx 1.2 \times 10^{-6}$ m/s. The near-bed acceleration $a = 2\pi U_w / T_p \approx 4 \times 10^{-7}$ m/s^2 is likewise negligible. With $V_c = 0.29$ m/s and $\beta = 0^\circ$, Eq. (10) gives $V_{design} \approx 0.29$ m/s, which is the velocity used in Eqs. (7)–(9) with the coefficients and seawater density in Table 2.

On-Bottom Stability and Geotechnical Scour

On-bottom stability under the ultimate limit state was verified using the DNV-RP-F109 generalised stability criterion, requiring the frictional resistance to exceed the amplified total horizontal hydrodynamic load:

$$\mu \cdot (WW_s - FF_L) \geq \xi \cdot (FF_D + FF_I) \quad (11)$$

with the seabed frictional resistance and resulting stability factor evaluated as:

$$FF_R = \mu \cdot WW_s \quad (12)$$

$$SF = \frac{FF_R}{FF_D} \geq 1.1 \quad (13)$$

Local scour and burial were assessed using the Shields mobility criterion, in which bed shear stress τ_b is compared against the critical Shields parameter:

$$\theta = \frac{\tau_b}{(\rho \rho_{sd} - \rho \rho_{sw}) \cdot g \cdot dd_{sd}} \quad (14)$$

$$SS_d = DD_{conc} \cdot (1.5\theta / \theta_c - 1)^{0.5} \quad (15)$$

where S_d is the predicted scour depth and the optimal burial depth is $hd = S_d + C_s$, with C_s a safety margin of 0.2–0.5 m to account for seabed variability.

Analytical Stress Formulations

The circumferential (hoop) stress governing wall-thickness design was evaluated as:

$$\sigma\sigma_h = \frac{(PP_i - PP_o) \cdot DD_m}{2 \cdot tt_{nom}} \quad (16)$$

Axial stress induced by restrained thermal expansion was evaluated as:

$$\sigma\sigma_t = E \cdot \alpha \cdot \Delta T \quad (17)$$

and the bending stress at mid-span, from the simply-supported maximum bending moment M_{max} , was evaluated as:

$$MM_{max} = (q + w) \cdot L^2 / 8 \quad (18)$$

$$\sigma_{\sigma_{bend}} = \frac{MM_{max} \cdot DD_o}{2 \cdot I} \quad (19)$$

where q is the hydrodynamic load and w is the submerged self-weight, both per unit length. The total longitudinal stress is the algebraic sum of the pressure, thermal, bending, pre-tension and end-cap components:

$$\sigma_{\sigma_L} = \sigma_{\sigma_p} + \sigma_{\sigma_t} + \sigma_{\sigma_{bend}} + \sigma_{\sigma_{pre}} + \sigma_{\sigma_e} \quad (20)$$

and the equivalent von Mises stress, used as the governing failure indicator, combines the hoop and longitudinal stress components:

$$\sigma_{\sigma_{eq}} = \sqrt{\sigma_{\sigma_h}^2 + \sigma_{\sigma_L}^2 - \sigma_{\sigma_h} \sigma_{\sigma_L}} \quad (21)$$

subject to the yield criterion $\sigma_{eq} \leq \eta_u \cdot \sigma_y$, with utilisation factor $\eta_u = 0.72$ per DNV-OS-F101 for the operational condition. The maximum permissible diameter-to-thickness ratio was checked against:

$$(D/t)(D/t)_{max} = 2 \cdot (\sigma_y / P P_i) \cdot \eta \eta_u \quad (22)$$

Eq. (22) is a pressure-containment (hoop) limit: with $\sigma_y = 450$ MPa, $P_i = 12$ MPa and $\eta_u = 0.72$ it gives $(D/t)_{max} = 54$. It therefore does not by itself fix the D/t limit reported in this study. The tighter limit in Section 4.3 comes from the combined bending, thermal and pressure stress computed by finite element analysis in the 30 m free span, and it depends on the span length, current velocity, support conditions and steel grade. In the code checks the hoop stress is evaluated on the internal gauge pressure, $\sigma_h = P_i \cdot D_o / (2t) = 192$ MPa, which is the basis used by ASME B31.4 because external pressure is not credited; the net-pressure form of Eq. (16) gives a hoop stress of only about 30 MPa at 1000 m water depth.

ASME B31.3 and B31.4 Stress Acceptance Criteria

Three ASME B31.3 stress categories were checked, each against a distinct allowable limit — sustained (primary), displacement (secondary) and occasional stress:

$$SS_l \leq SS_h \times \eta \eta_1 \quad (23)$$

$$SS_e \leq SS_a = f(1.25SS_c + 0.25SS_h) \times \eta \eta_2 \quad (24)$$

$$SS_{occ} \leq 1.33SS_h \times \eta \eta_3 \quad (25)$$

ASME B31.3 (Process Piping) and ASME B31.4 (Pipeline Transportation Systems for Liquids and Slurries) are distinct codes and are applied here for different purposes. B31.3 is a piping-stress code; its sustained, displacement and occasional categories (Eqs. 23–25) are used as a screening framework with the 300 MPa allowable, which is two-thirds of σ_y . B31.4 is the code for liquid pipelines, including offshore sections, and two of its limits are checked here: hoop stress not exceeding $0.72\sigma_y$ (324 MPa) and the equivalent stress of a restrained line not exceeding $0.90\sigma_y$ (405 MPa), per ASME B31.4 (§402.3.1 for hoop stress and §402.3.2 for the restrained-line equivalent stress). Neither code replaces DNV-OS-F101, which is the governing offshore pipeline standard for the limit-state checks in this study, and DNV-RP-F105 (free-span VIV assessment) was not applied (Section 4.6).

Lateral and Upheaval Buckling Stability

The effective compressive axial force, the principal driving force for pipeline buckling, was evaluated as (Wang, Shi & Yuan, 2023):

$$P = EA\alpha\Delta T + (1 - 2\nu) \cdot \frac{\pi}{4} D^2 \Delta P \quad (26)$$

The characteristic lateral buckling wavelength, dimensionless buckling parameter ΦL and empirical stability factor ΦW were then evaluated as:

$$L = (72 \cdot \mu EI / WW_s)^{1/4} \quad (27)$$

$$\Phi \Phi_L = L \cdot \sqrt{P/EI} \quad (28)$$

$$\Phi \Phi_W = (5.68/\Phi^2) - (88.35/\Phi^4) \quad (29)$$

from which the allowable axial load W_a , the maximum compressive force the pipeline can sustain without lateral buckling, follows as:

$$WW_a = \Phi \Phi_W \cdot \delta \delta^2 / EI \quad (30)$$

For upheaval buckling, the Hobbs (1984) critical axial force and the uplift resistance provided by soil cover were evaluated as:

$$PP_{cr} = CC_1 \cdot [W^2 \cdot E / L^2]^{1/3} + CC_2 \cdot \mu \cdot WW_s \cdot L / (m + 1) \quad (31)$$

$$q = H \cdot DD_o \cdot (1 + fH/DD_o) \cdot \rho \rho_{sg} \quad (32)$$

Fatigue Assessment and Cross-Validation

Fatigue life was assessed using Miner's linear cumulative damage rule, with the cumulative damage ratio D required to remain below 1.0 over the 25-year design life:

$$D = \sum_i \frac{n_i}{N_i} \leq 1.0 \quad (33)$$

using the DNV-RP-C203 S-N curve and stress range from the wave scatter diagram:

$$\log(N) = \log(\bar{a}) - m \cdot \log(\Delta\sigma) \quad (34)$$

$$\Delta\sigma \Delta\sigma_i = 2 \cdot SCF \cdot \sigma \sigma_{bend} \quad (35)$$

Finally, the finite element results were cross-validated against the closed-form analytical predictions using:

$$\varepsilon \varepsilon_{rel} = |\sigma \sigma_{FEA} - \sigma \sigma_{analytical}| / \sigma \sigma_{analytical} \times 100\% \leq 5\% \quad (36)$$

A discrepancy exceeding 5% would trigger a mesh refinement study and re-evaluation of the boundary-condition assumptions; this threshold was met throughout the present analysis.

RESULTS AND DISCUSSION

Here, the results of the analytical calculations and the ANSYS finite element simulations performed according to the methodology described in Section 3 are presented. Firstly, the key computational results are summarized in tabular form (Table 3) followed by detailed discussion with graphical interpretation of the data.

Result parameter	Value	Code / basis
Peak von Mises stress, σ_{eq}	278 MPa	ANSYS FEA
ASME B31.3 allowable stress	300 MPa	ASME B31.3

Stress utilisation ratio	0.927 (278/300)	ASME B31.3
ASME B31.4 combined stress utilisation	0.686 (278/405 MPa)	ASME B31.4, $0.90\sigma_y$
DNV-OS-F101 equivalent stress utilisation	0.858 (278/324 MPa)	DNV-OS-F101, $\eta_u = 0.72$
Hoop stress, σ_h (utilisation)	192 MPa (64.0%)	$P_i \cdot D_o / (2t)$, gauge pressure (Sec. 3.7)
ASME B31.4 hoop stress utilisation	0.593 (192/324 MPa)	ASME B31.4, $0.72\sigma_y$
On-bottom stability safety factor, SF	2.05	DNV-RP-F109 (min. 1.1)
External pressure vs collapse pressure	10.06 MPa vs 15.8 MPa (63.7% utilisation)	DNV-OS-F101
Cumulative fatigue damage ratio, D	0.67 (25-yr design life)	Miner's rule
Diameter-to-thickness ratio, D/t	32 (design point; 300 MPa reached at $D/t \approx 36$)	FEA, Fig. 5 (this configuration only)
Buckling parameter / stability factor, $\Phi L / \Phi W$	3.48 / -0.13	Eqs. (28)–(29)
FEA-analytical cross-validation error, ϵ_{rel}	< 5%	Eq. (36)

Table 3. Summary of key analytical and finite element results.

Material Selection and Stress Response

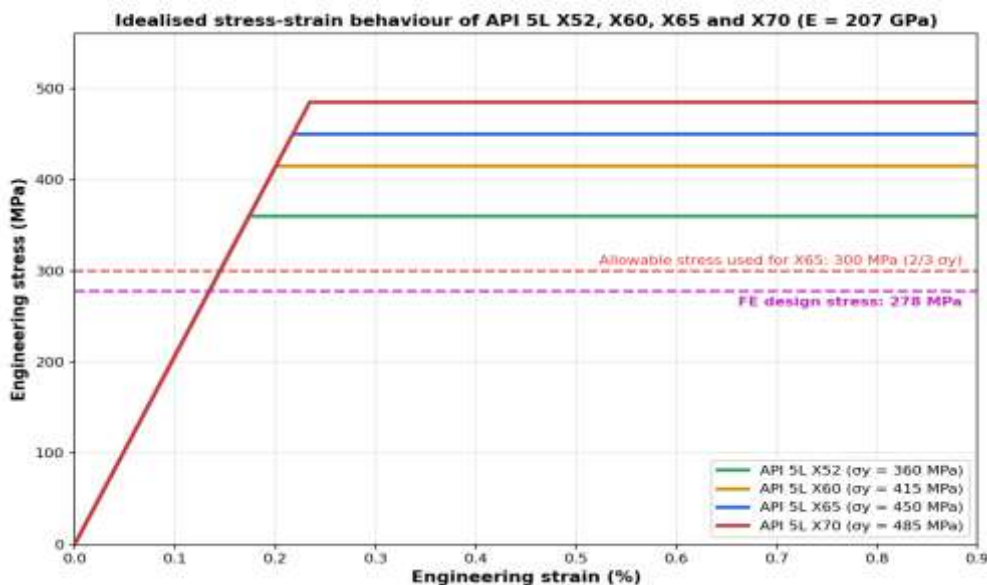


Figure 1. Idealised stress-strain behaviour of API 5L X52, X60, X65 and X70 steels (nominal specified minimum yield strengths), with the finite element design stress (278 MPa) and the 300 MPa allowable stress used for X65.

A comparison of the engineering stress-strain behavior of API 5L X52, X60, X65 and X70 (specified minimum yield strengths of 360, 415, 450 and 485 MPa, Figure 1) showed that all four grades share the same elastic slope of about 207 GPa and differ only at their respective yield points. The finite element design stress of 278 MPa

lies within the elastic range of all four grades, with a margin to yield of 82 MPa for X52, 137 MPa for X60, 172 MPa (38% of the yield strength) for X65 and 207 MPa for X70. The 450 MPa yield strength used in the finite element model and in Table 2 is that of API 5L X65, which is the grade analysed throughout this paper. Applying the same two-thirds rule that gives the 300 MPa allowable for X65, the allowable stress is 240 MPa for X52 and about 277 MPa for X60, both below the 278 MPa design stress, and about 323 MPa for X70. X65 is therefore the lowest of the four grades that satisfies the screening check for this loading. X70 would give more margin, but at a higher material cost and with greater welding difficulty for subsea girth welds, which were not assessed in this study.

Environmental Loading and Sensitivity

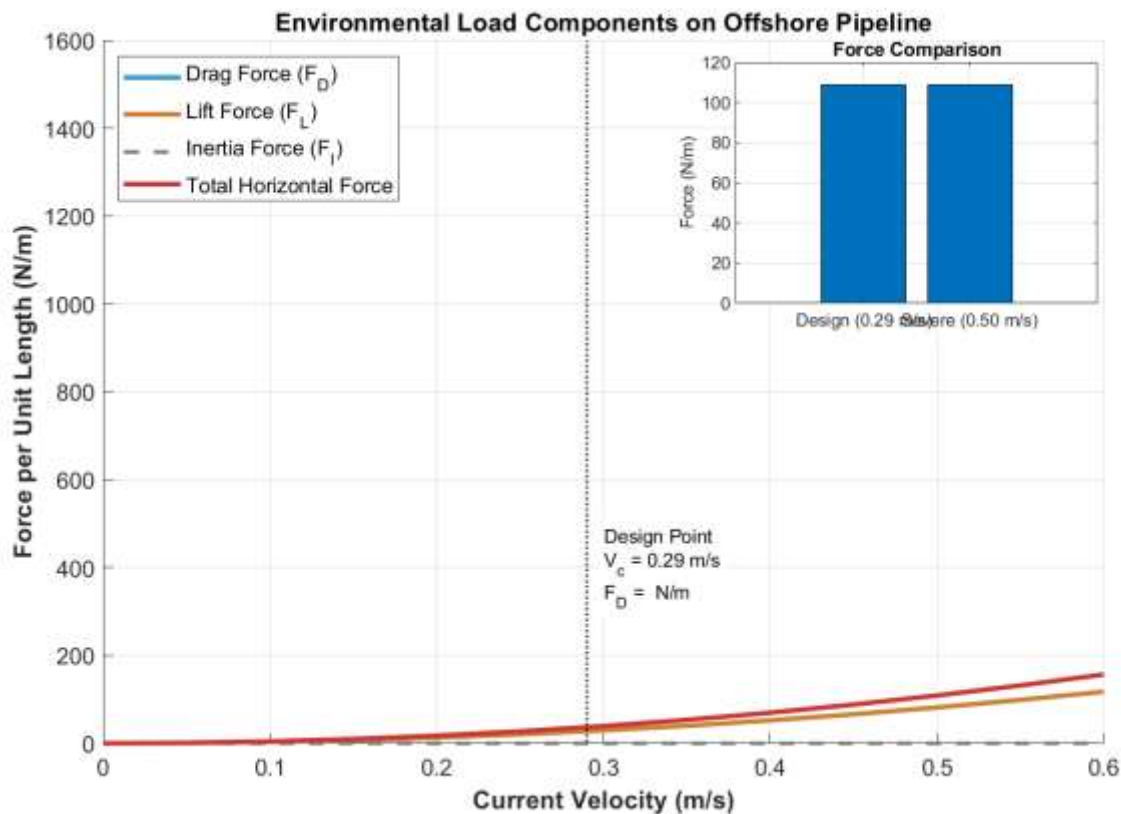


Figure 2. Environmental load components (drag, lift, inertia) at different current velocities.

The horizontal load at the 100-year design current velocity of 0.29 m/s is dominated by drag force at 412.5 N/m (69% of the total), and inertia at 188.7 N/m (31%), with lift reducing the effective submerged weight by 275 N/m. With the current velocity increasing by 72% to 0.50 m/sec, the drag force increased by about 198% (a factor of 2.98) to 1,227.5 N/m, consistent with the quadratic dependence of drag on velocity ($F_D \propto V^2$, since $(0.50/0.29)^2 = 2.97$). This identifies current velocity as the environmental parameter to which the drag load is most sensitive in this study (only the current velocity was varied in this check; wave height and period were not), and indicates that accurate, site-specific current data is critical for reliable design, particularly for routes exposed to variable current regimes.

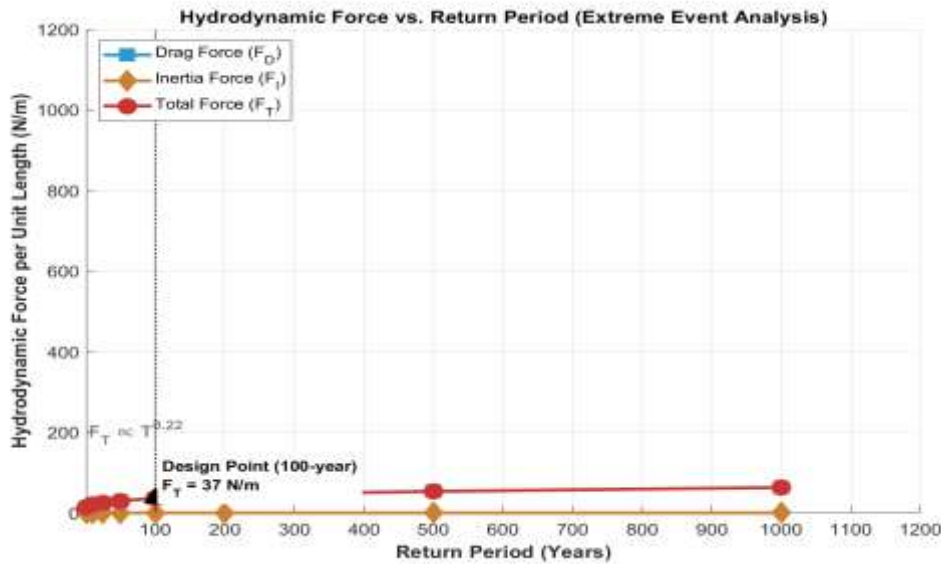


Figure 3. Hydrodynamic force versus return period for extreme-event analysis.

The total hydrodynamic force follows an approximate power-law relationship with return period ($F_T \propto T^{0.45}$), rising from around 250 N/m at the 1-year return period to 661 N/m at the 100-year design condition and approximately 1,050 N/m at 1,000 years. Because no natural plateau exists, the 100-year return period represents the accepted industry balance between structural robustness and economic feasibility, since the probability of a 1,000-year event occurring within the 25-year design life is only 2.5%.

Stress Distribution and Diameter-to-Thickness Effects

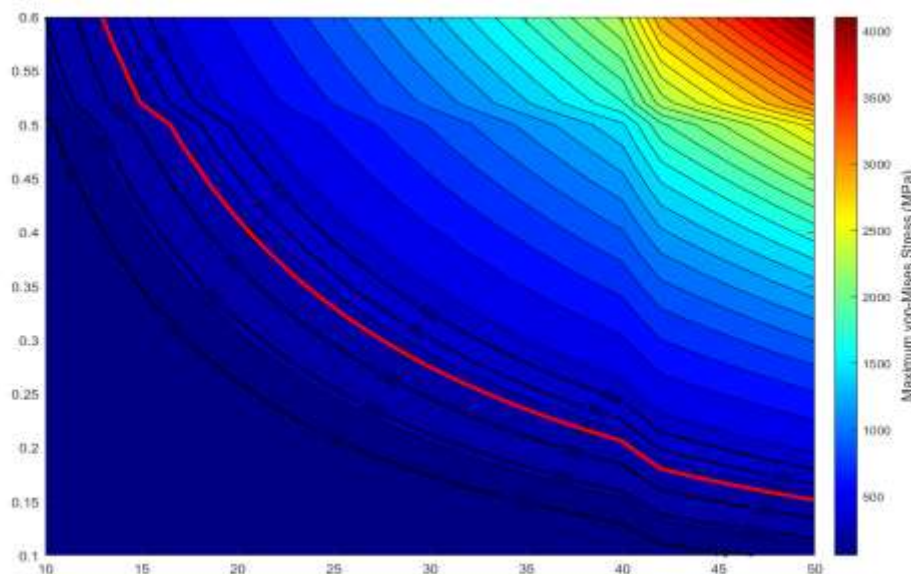


Figure 4. Combined effect of free-span length and current velocity on peak von Mises stress.

The combined effect of free-span length and current velocity on peak von Mises stress was mapped across the design space; spans less than 25 m and current velocities less than 0.35 m/s remain below 200 MPa, with the design point (30 m span, 0.29 m/s) sitting at approximately 278-300 MPa, close to the 300 MPa code-allowable contour. Due to the non-linear coupling between the span length and the current velocity, moderate increases in both can take a design from safe to critical. Therefore, the mapped contours can provide a practical decision-support tool for route planning and span-intervention scheduling. The contours were generated for one pipe (508 mm outer diameter, X65, 15.88 mm wall, fixed supports) and must be regenerated for other configurations.

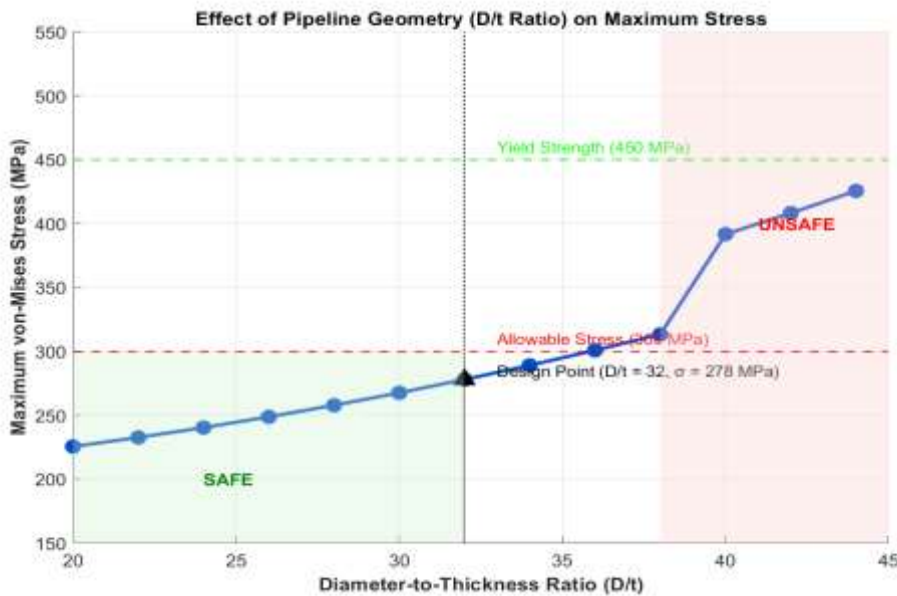


Figure 5. Effect of diameter-to-thickness ratio on maximum von Mises stress for the analysed 508 mm X65 pipeline (30 m span, 0.29 m/s current); the 300 MPa allowable is reached at D/t of about 36.

The D/t ratio was confirmed to be an important factor in structural integrity. For the analysed case (508 mm outer diameter, X65, 30 m span, 0.29 m/s current, fixed supports), the peak von Mises stress rises almost linearly from about 226 MPa at D/t = 20 to 278 MPa at D/t = 32, reaches the 300 MPa allowable at D/t of about 36, increases steeply beyond D/t = 38 (about 392 MPa at D/t = 40) and reaches about 425 MPa at D/t = 44, which is still below the 450 MPa yield strength within the plotted range. The design D/t of 32 (508 mm outer diameter, 15.88 mm wall) therefore lies inside the safe window, with a peak stress of 278 MPa against the 300 MPa allowable (utilization 0.927), and the allowable-stress limit for this configuration is D/t of about 36. D/t = 32 is the design point of this study and not a general limit. Three things set the limit: the stress is governed by bending in the 30 m span and not by hoop stress (the pressure-containment limit of Eq. (22) is D/t = 54); the 300 MPa allowable is tied to the 450 MPa yield strength of X65; and the loading is the 100-year current of 0.29 m/s. The result is valid over the D/t range plotted in Figure 5 (20 to 44), the span lengths and current velocities in the range of Figure 4 and the boundary conditions of Section 3.4. Extending it to other diameters, steel grades, span lengths, support stiffness or more severe currents requires new analysis, because the bending stress depends on the span length and the hydrodynamic load.

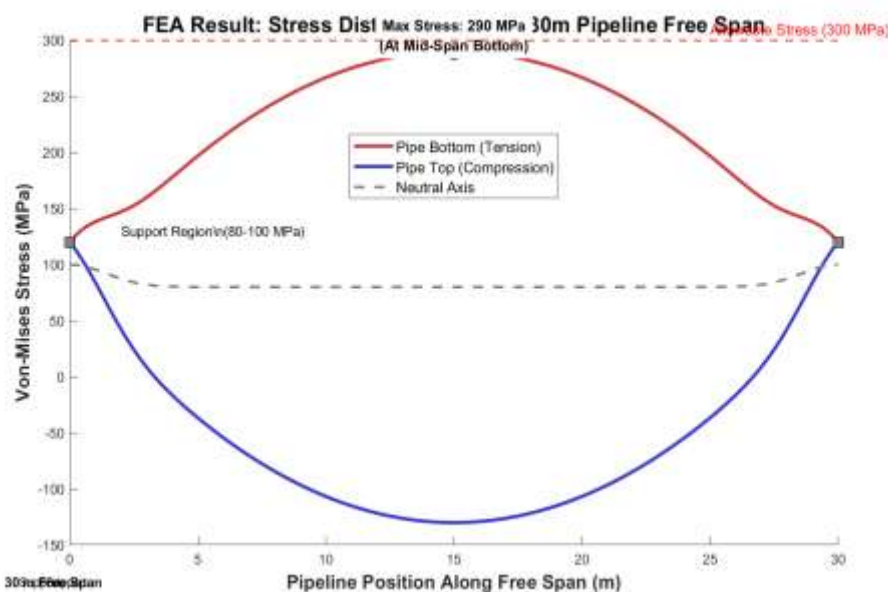


Figure 6. Von Mises stress distribution along the 30 m pipeline free span.

The stress distribution along the 30 m free span is characterized by a symmetric parabolic distribution typical for a fixed-fixed beam, with values of 80–100 MPa near the supports increasing to a maximum tensile stress of 278 MPa at the mid-span bottom fiber and a compressive stress of about –265 MPa at the top fiber, and a 543 MPa top-to-bottom gradient confirming that bending dominates the near-constant 80 MPa membrane stress induced by the internal pressure. This spatially resolved result identifies the mid-span bottom fiber as the governing inspection location, and demonstrates the added value of high-fidelity finite element analysis over simplified analytical estimates.

Fatigue and Buckling Stability



Figure 7. Cumulative fatigue damage accumulation over the 25-year pipeline design life.

The cumulative fatigue damage ratio calculated using Miner's rule over the 25-year design life is 0.22 at year 10, 0.52 at year 20 and 0.67 at year 25. This is below the threshold of 1.0 and corresponds to a calculated fatigue life of about 37 years under the same loading. It is a screening result and must be read with the limitations of Section 4.6: it uses a single S-N curve class, a representative wave scatter diagram and wave-induced stress ranges only, excludes vortex-induced vibration in the free span, and is compared with an unfactored limit of 1.0. Design standards apply a design fatigue factor (DFF) so that the allowable damage becomes 1/DFF; for a DFF of 3 the allowable would be 0.33 and the calculated 0.67 would not satisfy it. The 33% margin to 1.0 should therefore not be read as a demonstration of fatigue compliance, and the check should be repeated with project-specific scatter data, joint-specific S-N data, a vortex-induced vibration assessment and the applicable design fatigue factor.

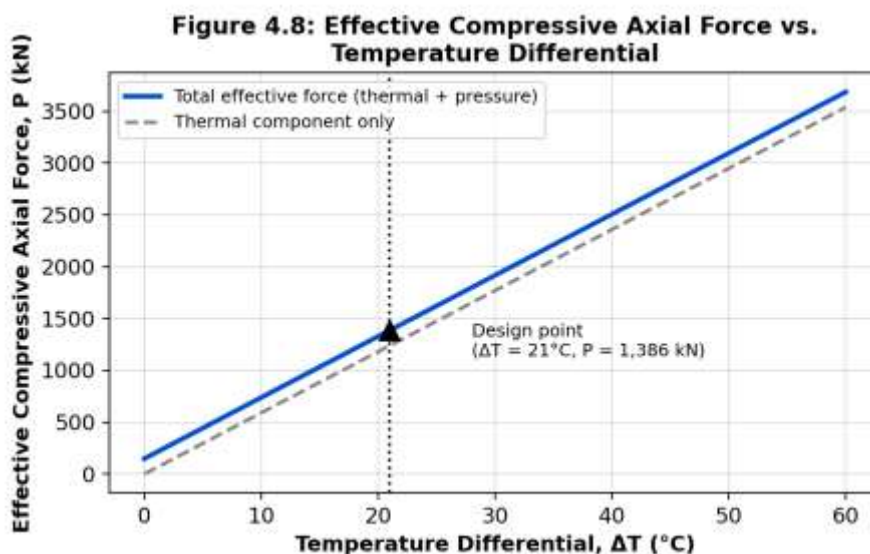


Figure 8. Effective compressive axial force versus temperature differential.

The resultant axial compressive force due to restrained thermal expansion and operating pressure was about 1,386 kN at the design temperature differential of 21°C (Figure 8). The thermal component is dominant, confirming (Eq. 26) that restrained thermal expansion is the main mechanism governing pipeline buckling in this design.

Figure 4.9: Lateral Buckling Wavelength and Buckling Parameter

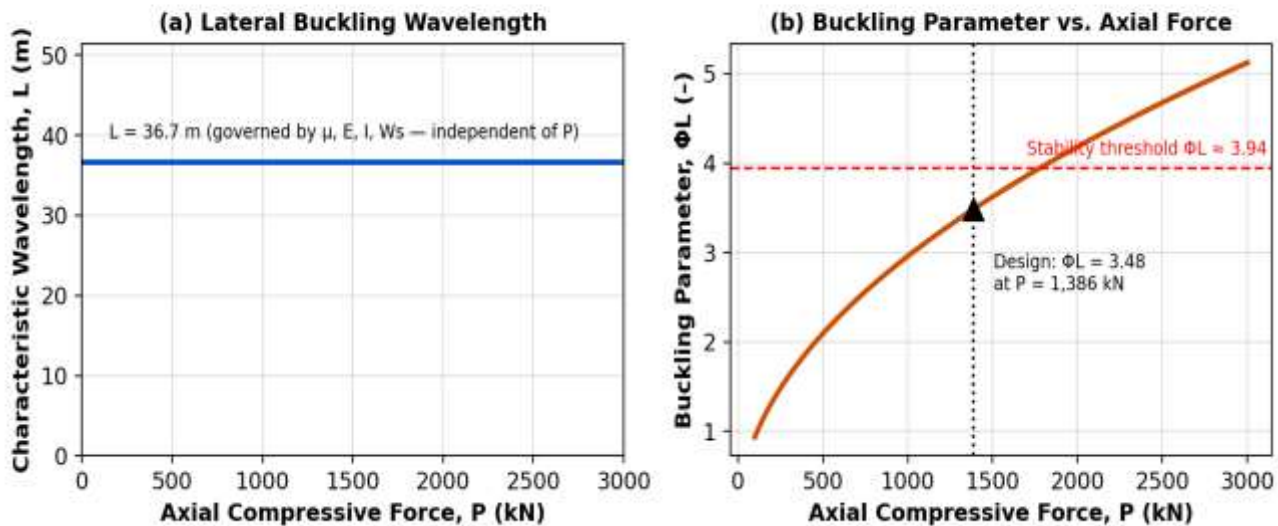


Figure 9. Lateral buckling wavelength and buckling parameter versus axial force.

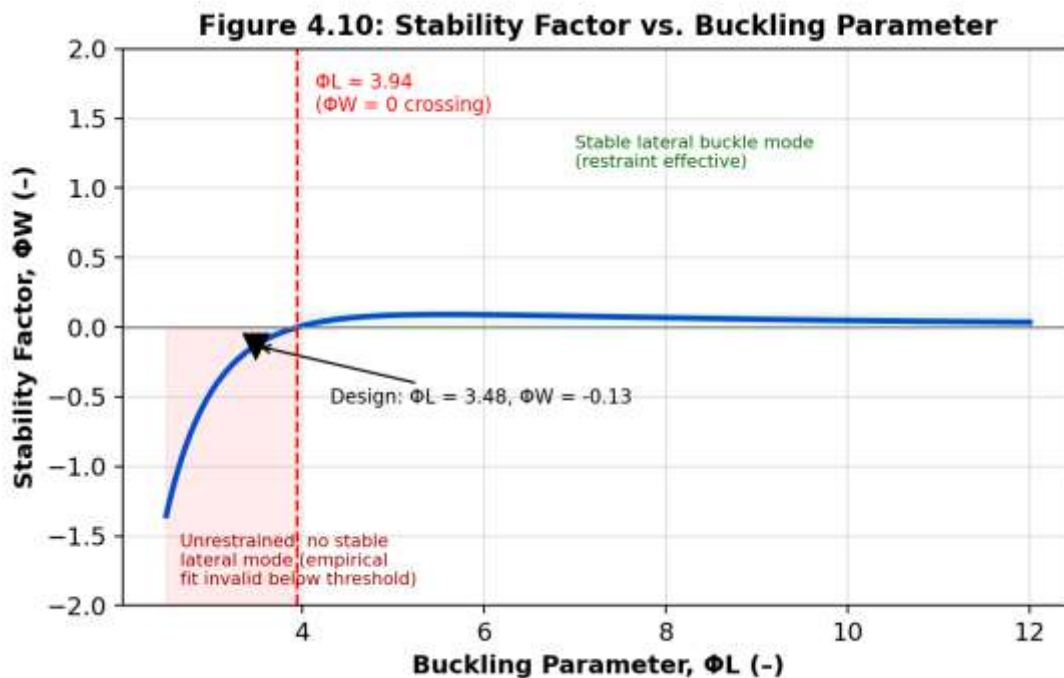


Figure 10. Stability factor versus buckling parameter.

The characteristic lateral buckling wavelength is a function only of the soil friction, flexural stiffness and submerged weight (Eq. 27). This was constant at 36.7 m and the buckling parameter ΦL increased to 3.48 at the design axial force. This is below the approximate 3.94 threshold where the empirical stability-factor correlation of DNV-RP-F109 (Eq. 29) changes sign. The corresponding stability factor, $\Phi W = -0.13$, is in the negative region, which means that the friction resistance alone is insufficient to ensure a stable lateral buckle mode at this axial force, and indicates a real engineering requirement for lateral restraint measures such as rock berms,

sleepers or engineered buckle initiators. These buckling results depend on the representative friction coefficient ($\mu = 0.6$) and would change with site-specific soil data (Section 4.6).

Figure 4.11: Upheaval Buckling — Critical Force and Uplift Resistance

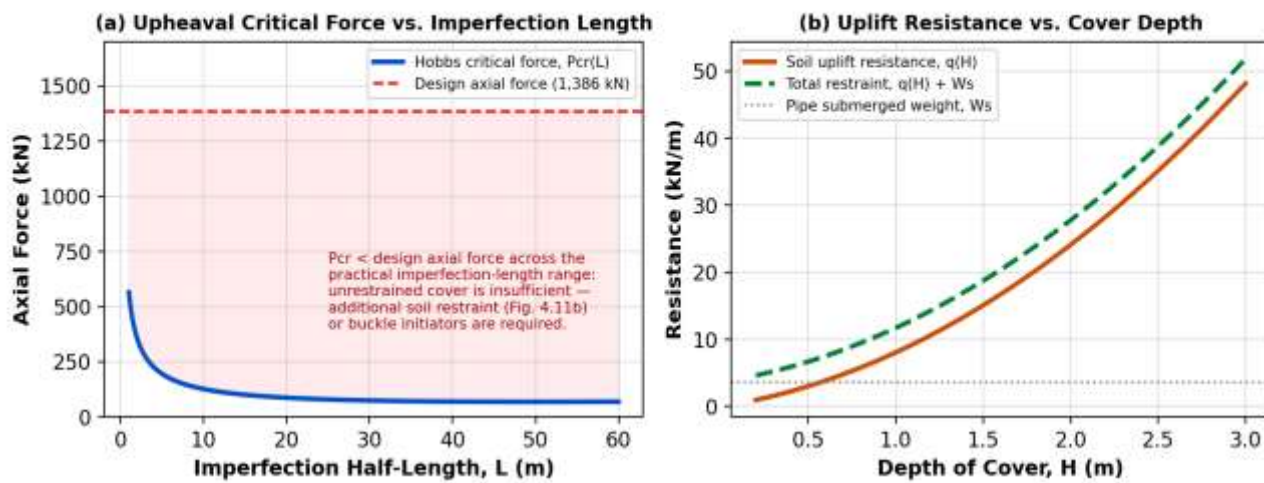


Figure 11. Upheaval buckling: critical force and uplift resistance versus cover depth.

For practical imperfection half-lengths greater than about 0.3 m, the 1,386 kN driving force was greater than the Hobbs critical force for upheaval buckling (Eq. 31), suggesting that the section is susceptible to upheaval at realistic imperfection sizes for an unrestrained or lightly covered section. The uplift resistance due to soil cover (Eq. 32) increases from about 1 kN/m at 0.2 m cover to about 48 kN/m at 3.0 m cover. This confirms that sufficient burial depth, and not pipe self-weight alone, is required to suppress upheaval where out-of-straightness cannot be avoided. The Hobbs critical force and the uplift resistance depend on the assumed imperfection size and on representative soil cover properties, so the cover depths quoted are indicative.

Code Validation Summary

Validation check	Applied value	Allowable limit	Utilisation
ASME B31.3 stress check	278 MPa	300 MPa	92.7%
Hoop stress check	192 MPa	300 MPa	64.0%
ASME B31.4 hoop stress check	192 MPa	324 MPa ($0.72\sigma_y$)	59.3%
ASME B31.4 combined stress check	278 MPa	405 MPa ($0.90\sigma_y$)	68.6%
DNV-OS-F101 equivalent stress check	278 MPa	324 MPa ($\eta\sigma_y$)	85.8%
Material yield check	278 MPa	450 MPa	61.8%
DNV-OS-F101 collapse check	10.06 MPa	15.8 MPa	63.7%
DNV-RP-F109 on-bottom stability check	SF = 2.05	SF $\geq$ 1.1	53.7% ($1.1/2.05$)
Deformation limit check (L/100)	< 300 mm	300 mm	within limit

Table 4. Code compliance validation summary.

For the analysed configuration the integrated framework satisfied every check in Table 4: the ASME B31.3 and ASME B31.4 stress limits, the DNV-OS-F101 equivalent-stress and collapse checks, the DNV-RP-F109 on-bottom stability check and the deformation limit. The finite element and analytical stress results differed by less than the 5% tolerance of Equation (36). The peak von Mises stress of 278 MPa is below the ASME B31.3 allowable of 300 MPa (utilization 0.927, a margin of 7.3%), below the ASME B31.4 combined-stress limit of 405 MPa (68.6%) and below the DNV-OS-F101 limit of 324 MPa (85.8%), and it reaches 61.8% of the 450 MPa yield strength of API 5L X65. The hoop stress of 192 MPa is 64.0% of the 300 MPa allowable and 59.3% of the ASME B31.4 hoop limit of 324 MPa. The applied external pressure of 10.06 MPa is 63.7% of the DNV-OS-F101 collapse pressure of 15.8 MPa; equivalently, the collapse pressure exceeds the applied pressure by 57%, and 36.3% of the collapse capacity is unused. These two figures describe the same result on different bases and are not additive. The ASME B31.3 stress screen has the highest utilization of all the checks and is therefore the governing check.

The Study's Limitations

The results should be interpreted within the following limitations, each of which affects how far the reported values can be relied on and is not only a topic for future work. First, the structural response was assessed under static and quasi-static loading; time-domain wave loading and vortex-induced vibration were not modeled. Free spans are susceptible to vortex-induced vibration, so the fatigue damage ratio of 0.67 and the free-span stresses are lower-bound estimates of the total response, and the span has not been shown to be acceptable under DNV-RP-F105, which is why that code is not listed among the checks completed. Second, nonlinear Beam-on-Elastic-Foundation springs calibrated to representative soft-clay parameters were used in place of site-specific geotechnical survey data. The friction coefficient, soil stiffness and cover resistance control the on-bottom stability factor, the lateral buckling wavelength and the upheaval buckling resistance, so these outcomes apply to the assumed soil and would shift with the spatial variability of a real route. Together with the fixed end supports, which underestimate the mid-span bending stress relative to compliant supports, this means that the 278 MPa peak stress and the 0.927 utilization are best estimates and not conservative bounds. Third, the metocean loading was based on the 100-year return period design condition, which is common practice; longer-term climate-driven changes in wave height and current velocity were not modeled, and because drag scales with the square of velocity (Section 4.2) a modest increase in current could consume the remaining 7.3% stress margin. Finally, the fatigue assessment used a single S-N curve class, a representative wave scatter diagram and no design fatigue factor, so the fatigue result is a screening value (Section 4.4). The stress, stability and fatigue values are therefore conditional on these limitations and are benchmarks for this configuration; they should be recalibrated with site-specific data before being applied to a particular field development.

CONCLUSIONS AND RECOMMENDATIONS

Conclusion

An integrated analytical and finite element procedure has been developed and applied to the stress and on-bottom stability analysis of a specific case: a 508 mm outer diameter API 5L X65 pipeline with a 15.88 mm wall in a 30 m free span at 1,000 m water depth in the Gulf of Guinea. The critical location was the mid-span bottom fiber, where the peak von Mises stress was 278 MPa. This gives an ASME B31.3 utilization ratio of 0.927 against the 300 MPa allowable (a 7.3% margin), an ASME B31.4 combined-stress utilization of 0.686, a DNV-OS-F101 equivalent-stress utilization of 0.858 and 61.8% of the specified yield strength. A 49,841-node, 24,786-element hexahedral mesh was used and mesh independence was verified.

The pipeline exhibited an on-bottom stability safety factor of 2.05 against the DNV-RP-F109 minimum of 1.1 under 100-year return period metocean loading. In the sensitivity study, in which only the current velocity was varied, a 72% increase in current velocity (0.29 to 0.50 m/s) produced an approximately 198% increase in drag force. The screening fatigue check gave a cumulative damage ratio of 0.67 over the 25-year design life against an unfactored limit of 1.0; it excludes vortex-induced vibration and applies no design fatigue factor, so it does not by itself demonstrate fatigue compliance. The finite element and closed-form analytical stress results agreed to within 5%.

The design diameter-to-thickness ratio of 32 lies inside the safe window for the analysed configuration, in which the 300 MPa allowable was reached at D/t of about 36. This is a case-specific result for the stated diameter, grade, span length, loading and support conditions, and not a general limit for X65 pipelines or for the Gulf of Guinea. The buckling analysis indicates the need for engineered lateral restraint and adequate burial depth at sites prone to lateral and upheaval buckling, rather than reliance on friction and self-weight alone. All values are conditional on the limitations in Section 4.6. They can support preliminary integrity screening of this configuration and serve as a starting point for comparable configurations only after recalibration with site-specific data.

Recommendations

Future work should extend the current static and quasi-static framework to full dynamic time-domain fatigue and vortex-induced vibration analysis, including a DNV-RP-F105 free-span assessment and a design fatigue factor, and explore the effects of climate-driven changes in wave height and current velocity on long-term pipeline stability in the Gulf of Guinea. Further, it is recommended that (i) site-specific geotechnical surveys be used to replace the representative soil parameters used here prior to application of the framework to a specific field development; (ii) engineered lateral restraint (rock berms, sleepers or buckle initiators) and a minimum burial depth consistent with Section 4.4 be specified at locations identified as prone to lateral or upheaval buckling; and (iii) project-specific wave scatter diagrams and joint-specific S-N data be used to refine the fatigue life estimate for detailed design.

Declarations

Acknowledgement

The Authors wish to thank the Department of Marine and Offshore Engineering, Faculty of Engineering, Rivers State University.

Data Availability Statement

The datasets generated and analyzed during the current study are available from the corresponding author on reasonable request, including the ANSYS Workbench model files and analytical calculation records.

Nomenclature

Symbol	Definition
A, A_{exact}	Cross-sectional area of pipe wall (approximate, exact) (m^2)
D_o, D_i, D_m	Outer, internal and mean pipe diameter (m)
D_{eff}	Effective outer diameter including coatings (m)
t, t_{nom}	Pipe wall thickness, nominal wall thickness (m)
I	Second moment of area (m^4)
Z	Elastic section modulus (m^3)
r_o, r_i	Outer and internal pipe radius (m)
V, m	Steel volume (m^3) and mass (kg)
E	Young's modulus (Pa)
ν	Poisson's ratio

σ_y	Specified minimum yield strength (Pa)
$\rho_{\text{steel}}, \rho_{\text{sw}}, \rho_{\text{sd}}$	Density of steel, seawater, sediment (kg/m^3)
α	Coefficient of thermal expansion ($^{\circ}\text{C}^{-1}$)
$\Delta T, \Delta P$	Temperature and pressure differential ($^{\circ}\text{C}$, Pa)
P_i, P_o	Internal and external design pressure (Pa)
ϵ_{mesh}	Relative discretisation (mesh-convergence) error
FD, FL, FI	Drag, lift and inertia force per unit length (N/m)
CD, CL, CM	Drag, lift and inertia coefficients
V, a	Design flow velocity (m/s), near-bed flow acceleration (m/s^2)
V _{design} , V _c , U _w	Combined design, steady current and wave orbital velocity (m/s)
β, k, h	Wave-current heading angle ($^{\circ}$), wave number (rad/m), water depth (m)
μ	Seabed friction coefficient
W _s	Submerged pipeline weight per unit length (N/m)
ξ	DNV-RP-F109 safety factor
FR, SF	Seabed frictional resistance (N/m), stability factor
θ, θ_c	Shields parameter, critical Shields parameter
τ_b	Bed shear stress (N/m^2)
S _d , h _d	Scour depth, optimal burial depth (m)
$\sigma_h, \sigma_t, \sigma_{\text{bend}}, \sigma_L, \sigma_{\text{eq}}$	Hoop, thermal, bending, combined longitudinal and von Mises equivalent stress (Pa)
M _{max}	Maximum bending moment at mid-span ($\text{N}\cdot\text{m}$)
q, w	Hydrodynamic and submerged self-weight load per unit length (N/m)
η_u	Code utilisation factor
D/t	Diameter-to-thickness ratio
S _I , S _e , S _{occ}	ASME B31.3 sustained, displacement and occasional stress
P	Effective compressive axial force (N)
L (buckling)	Characteristic lateral buckling wavelength (m)
Φ_L, Φ_W	Buckling parameter, stability factor (dimensionless)
W _a , δ	Allowable axial load (N/m), permissible lateral deflection (m)

Pcr, C1, C2	Hobbs critical upheaval buckling force (N), empirical constants
q (uplift)	Uplift soil resistance per unit length (N/m)
H, f, ρ_{sg}	Depth of cover (m), uplift resistance coefficient, submerged density of cover (kg/m ³)
D (fatigue), ni, Ni	Cumulative fatigue damage ratio, applied cycles, cycles to failure
$\Delta\sigma$, SCF	Stress range (MPa), stress concentration factor
ϵ_{rel}	Relative FEA-analytical cross-validation error

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