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1 In-Situ Quasi-Static and Dynamic Behavioural Response of Steel Tubular Frames Subjected to Lateral Impact Loads

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Abstract

Steel tubular members are widely used as primary and secondary structural framing members in offshore oil and gas platforms. A platform is inherently liable to collisions from ships which can create severe structural damages in the rig. The effect of this damage has been studied by a number of researchers through investigating the impact behaviour *isolated* tubular members. This is while, the *in-situ* response of a member located in a structural frame, to lateral impact loads, is not necessarily the same as the response of an individual *isolated* impacted member. In this paper the behaviour of a chord member forming part of a tubular frame, subjected to impact loads, has been investigated. The tubular frame was tested experimentally by other researchers and reported in the literature. The non-linear numerical models of the frame presented by the authors have been validated against the experimental results. These validated models have been examined under both quasi-static and dynamic impact loads with operational pre-loading applied. It has been found that, in a pre-loaded frame, quasi-static impact loading results in the failure of the impacted member. Interestingly, dynamic modelling of the impact results in the dynamic instability of an adjacent bracing member. It has been noticed that, under a dynamic impact, the impacted *in-situ* member (located in the frame) behaves rather similarly to a pin ended *isolated* member. With a quasi-static impact, the impacted *in-situ* member follows fairly closely the response obtained for a fixed ended *isolated* member.

Keywords

Impact, Frame, Tubular, Dynamic, Quasi-static and Failure

2 NOMENCLATURE

σ_y	Material yield stress
D	Tube diameter
E	Material modulus of elasticity
F	Concentrated lateral impact load
F_o	Dynamic lateral step load
$F_P=8D^2t\sigma_y/L$	Plastic collapse load of a tubular beam in pure bending
I	Moment of inertia of the cross-section
L	Tube length
$\bar{m}$ m	Mass per unit of length
P	Push over load
$P_y=\pi Dt\sigma_y$	Axial squash load of the tube
P_u	Ultimate axial load of the tube
R	Tube radius
t	Tube wall thickness
T	Natural period of vibration

1 INTRODUCTION

Design of offshore structure components against a ship collision is generally based on available knowledge of the behaviour of damaged and impacted *isolated* tubular members. The majority of previous studies presented in the literature on ship-offshore collisions concentrate on the behaviour of *isolated* tubular members (Zeinoddini *et al.* [21, 22]). The behaviour of an impacted member when it is part of a structural frame is not necessarily the same as the behaviour of the corresponding *isolated* member, remote from the framework, subjected to a similar impact load.

The differences between the response of a tubular member which is part of a structural frame (*in-situ* member) and an *isolated* member, when both are subjected to similar impact loads, are caused by several parameters. Interaction between the global modes of vibration in the frame and the member modes of deformation of the impacted member can create changes in the member response. In addition, the inertia forces and damping effects in the structural frame are different to those of an *isolated* member. The boundary conditions of an impacted *in-situ* member are a function of the connection properties and the stiffness of other members meeting at the connection. This type of semi-rigid boundary condition is different from the typical rigid or free end conditions considered in the literature for the study of *isolated* tubular members (Yao *et al.*[19]; Frieze and Cho[6]; Amdahl and Eberg[3]; Rambech and Dahl[12]; and Ricles and Bruin[13]). The boundary conditions of an *in-situ* impacted member are also likely to change during the impact. This occurs as a consequence of deformations in other members and in the connections.

The main objective of the current study is to investigate the difference between the response of an impacted member which is part of a structural frame (an *in-situ* member) and the response of a similar *isolated* member. Such a study can link knowledge available in the literature on the behaviour of impacted and damaged *isolated* tubular members to *in-situ* response which is closer to real behaviour.

2 THE BENCH MARK TUBULAR FRAME

2.1 Test Programme

Structural frames have been tested experimentally by other researchers under vertical loads, lateral loads and base excitations (Martin and Villaverde[9]; and Mosalsm *et al.*,[10]). However, the authors of the current study are not aware of any relevant impact testing of tubular frames which can represent ship collisions with offshore structures. Large-scale tubular frames have also been tested under static push over loading by other researchers to study the failure of jacket frames (Grenda *et al.*[7]; Nichols *et al.*[11]; and Bolt[4]).

The bench mark tubular frames mentioned above were found to be the most relevant available experimental work that could be used for validating the numerical impact model. The experimental results from these bench mark tubular frames have been used in this study for validating the non-linear numerical models of the frame with push over loading. If the non-linear numerical model can correlate with a push over collapse experiment, this provides some basis for using the model for the prediction of the dynamic behaviour, although it must be admitted that the accuracy of the some aspects of dynamic modelling remains uncertain.

The experimental data, which were used for benchmarking the non-linear finite element models of tubular frames, emanate from the Phase I Frames Test Programme carried out in the placecountry-regionUK. This experimental push over test project was initiated in 1987. The programme was conducted by Billington Osborne-Moss Engineering Limited (BOMEL) as part of a joint industry programme with the object of providing test data on the collapse behaviour of jacket structures and in addition, to develop calibrated software for the non-linear push over analysis of framed structures (Bolt *et al.*[4]). The Phase I Frames Test Programme was completed in 1990 and provided the first large-scale test data on the collapse performance of frames representative of offshore structures. The results of this programme were released from confidentiality in 1993 (Lalani *et al.*[8]).

The Phase I Frames Test Programme consisted of testing four, two bay, X-braced frames. These tubular frames (*Figure 1*) were the largest frameworks ever to be tested to collapse in a controlled manner, and provided a new and important insight into the role of redundancy and particularly tubular joint failures within a frame, neither of which have been investigated in earlier research programmes (Bolt *et al.*[25]).

2.2 Numerical Models of the Bench Mark Frame

The ABAQUS[1] non-linear finite element program has been used to produce two identical numerical models of the bench mark frame 1. In the first numerical model, each vertical, horizontal and diagonal member of the frame shown in *Figure 1* has been modelled using up to 20 beam elements (type *PIPE31*). The connections are considered to be rigid and the frame supports are pinned. The second numerical model is the same as the first except that one of the chord members in the upper bay has been modelled using shell elements. Twenty four shell elements (*S4R*) have been used in the circumference and fifty in the longitudinal direction of the chord member. The circumferential nodes at the two ends of this member have been linked to the end node of their adjacent beam element using the multi-point constraint option available in ABAQUS.

The second model allows for local deformations in the chord member which is important in a ship impact study. The first numerical model excludes local deformation and denting. Comparing the results obtained from the first and the second models reveals the effect which local deformations have on the response of the frame.

The first model, using only beam elements, is similar to models which have been used in the design against ship collision for the majority of existing offshore structures. The limited past capacity of computational facilities did not allow a time consuming analysis capable of including local effects in the appraisal (Sterndorff *et al.*[16]; and Waegter and Sterndorff[18]). It should be mentioned that no imperfection has been considered in the above mentioned numerical models.

2.3 Validation of Numerical Models

Figure 2 shows the horizontal load-displacement curves from the experimental results and in addition from the two numerical models when the frame is subjected to a push over horizontal load at its top. It can be seen that there is a good agreement between the test and the numerical results. Under push over loading no difference was found in the response of both of the numerical models. As a result, one curve in *Figure 2* represents the response of both numerical models, with and without local deformations included.

Buckling of the compression brace at the top half of the upper bay was reported to have caused failure in the test specimen. The same phenomenon was observed to have occurred in the numerical models. The ultimate lateral capacity of the frame tested was found to be 920kN. The lateral capacity predicted by both the numerical models was found to be 932kN.

3 RESPONSE TO QUASI-STATIC IMPACT LOADS

3.1 Models of the Isolated Impacted Tubular Members

The two numerical models of the test framework have been examined under lateral impact loads applied at mid length of a chord member in the frame upper bay. To be able to compare the results from the *in-situ* tubular members to the response of impacted *isolated* tubular members, two *isolated* chord members have also been modelled. Again in one model shell elements have been used which allow for the inclusion of local effects while in the other numerical model beam elements have been employed which model bowing but do not model local denting deformation behaviour.

As a result, four numerical models have been considered; namely a frame with local effects; a frame without local effects; an *isolated* chord member with local effects and an *isolated* chord member without local effects. The geometry and other properties of the *isolated* chords are

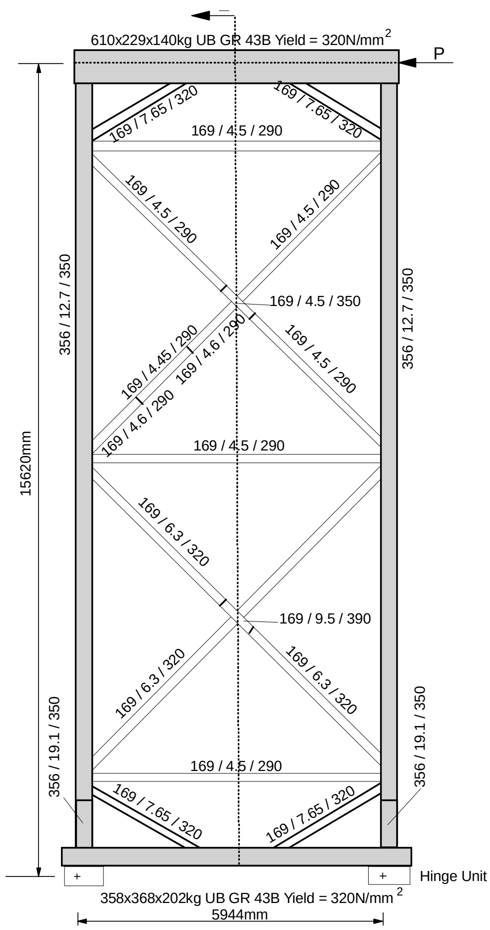

Figure 1 Elevation and properties of the tubular frame (frame 1),used in the benchmarking exercise[1] (Nichols *et al.*, 1994)

the same as those of the impacted member in the frames.

The boundary conditions at both ends of the *isolated* chord members allow for free axial sliding at each end of the member. However, ends are completely restrained against rotation. This boundary condition models, as closely as possible, the boundary conditions of the impacted member in the test frame although some axial restraints will be present.

3.2 Pre-Loading

A ship collision usually occurs when an offshore structure is carrying its operational load. The gravity and operational loads (called here pre-loading) can exacerbate the level of structural damage caused by a collision. With design safety factors, the operational pre-loading can be assumed to be, at maximum, between 50% and 70% of the ultimate axial load of the member (P_u). The former is the more likely load for a collision event and the latter is mostly associated with severe environmental load conditions. In this study only vertical pre-loading in the frame

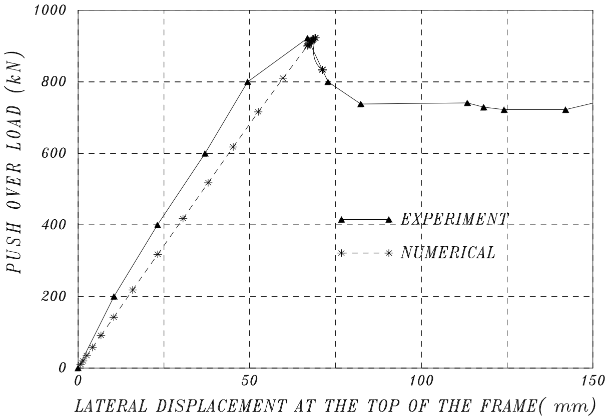

Figure 2 Lateral push over load-displacement curves for test frame 1.

members (or axial compression in the *isolated* chord members) has been considered. The pre-loading in all models produces an axial compression in the chord member equal to 50% of the axial squash load of the member (P_y).

3.3 Response of Pre-Loaded Tubulars to Quasi-Static Impacts

The four main numerical models have been studied under a quasi-static lateral impact load applied at the mid-span of the chord member. A Modified Riks method of analysis has been used because an impacted member experiences local and/or global instabilities in the tube wall or in overall. In non-linear finite element analysis, the Modified Riks Method is used for unstable static problems such as those found in post-collapse or post-buckling behaviour. The tangent stiffness matrix can be examined at any stage of the loading to determine the existence of negative eigenvalues. These can then be used to define the type of instability occurring in the structure. *Figure 3* shows the response of the four main numerical models to a quasi-static impact load. The ordinate represents the dimensionless lateral load. The abscissa in *Figure 3* shows the dimensionless lateral deformation at the position of the impact load.

When local effects are included in the numerical model, the maximum lateral load which can be resisted by the *in-situ* member is about 40% less than the maximum load carried when local effects are ignored. The ratio of corresponding values for the *isolated* chord member is about 33% with the model including local effects producing the lower value (*Figure 3*). The figure underlines the importance of including local effects in a collision study.

Both *isolated* and *in-situ* chord members support similar maximum lateral loads when the local effects are excluded. The similar values for the maximum lateral loads obtained for the *isolated* and *in-situ* members indicate that with quasi-static impacts the rigid boundary conditions, considered for the *isolated* chord member, are fairly close to the real circumstances found in the structural frame.

In the numerical models where local effects have been included, the peak load in the *isolated* member is slightly higher than the peak load obtained for the *in-situ* member. Reduction in

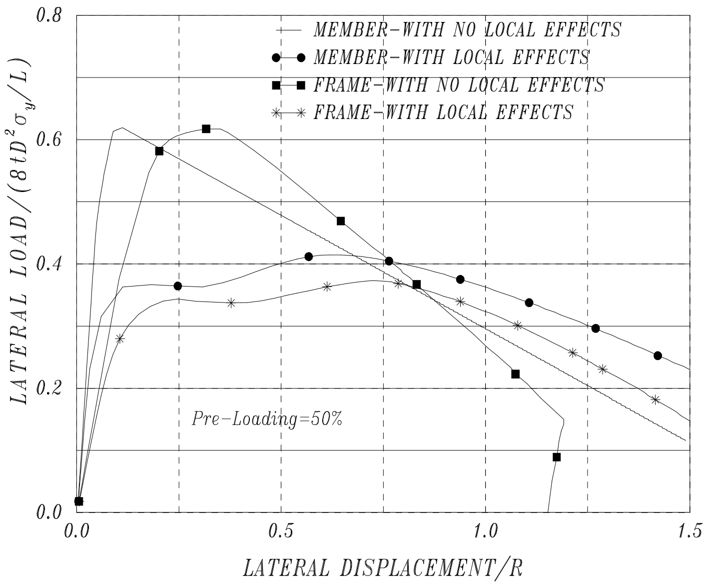

Figure 3 Lateral load-displacement behaviour of the *isolated* and *in-situ* tubular chord members subjected to lateral quasi-static impact.

the lateral load capacity indicates that the two ends of the damaged *in-situ* chord member exhibit a semi-rigid behaviour compared with the fully restrained condition for the *isolated* member. It will be seen later that with dynamic impact loads, considerably higher levels of semi-rigidity appear at the ends of the *in-situ* members.

4 RESPONSE TO DYNAMIC IMPACT LOADS

The four numerical models of frames and *isolated* members outlined in Section 3.1 have been, this time, examined under dynamic impact loads. As for previous models a pre-loading has been applied with all models and then lateral impact has been applied. An implicit incremental direct integration approach, based on the 'Newmark method', has been applied using the finite element program ABAQUS. In order to acquire a pure response which is not affected by modification of external loads and inertia forces during the impact, step lateral loads have been used initially hence providing a better understanding of the dynamic characteristics of the impact.

In the current study no structural damping has been incorporated into the finite element models. The only existing damping in the analysis is the numerical Hilber-Huge damping incorporated in the finite element program. This damping is relatively small compared to structural damping. It should be mentioned that

Obviously, some structural damping forces are involved in the response of the tubes to a dynamic load, but damping forces are generally believed not to impose a significant influence during the impact. This is because the duration of the impact is usually very short. In ordinary structures, impact forces need a duration several times greater than the fundamental natural period of the system to have a remarkable effect on the response (Zeinoddini *et al.*[25]).

4.1 Stable Responses

Figure 4 shows the response of the four main numerical models when a dynamic step lateral load of $160kN$ ($=0.22[8tD^2\sigma_y/L]$) is applied at the mid-span of the chord member. Time histories of the deformation at the mid-span of the impacted chord member are displayed in this figure. All the models carry a pre-loading which produces an axial compressive force equal to 50% of the squash load in the chord member.

It can be seen in *Figure 4* that with 50% pre-loading and the 160 kN lateral step load, all the responses remain bounded and therefore stable. Models with local effects show larger displacements, essentially due to the denting of the chord wall. Some small fluctuations can be observed in the response of the models where local deformations have been included. The responses of the models where local deformations have been omitted are smooth and free of these small fluctuations. The fluctuations appear to be produced by the excitation of higher modes of vibration within the tube wall.

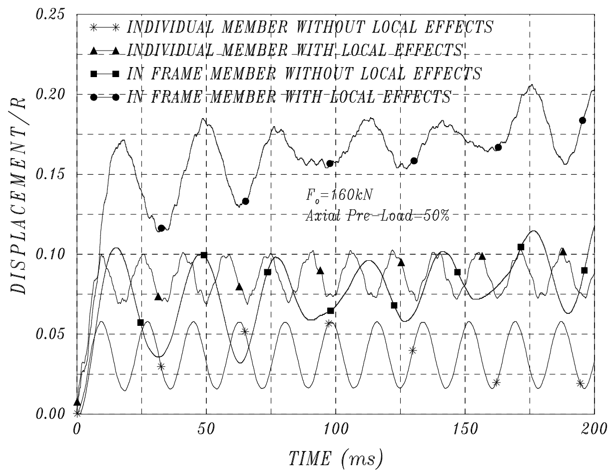

Figure 4 Time history of lateral deformation of the tubular frames and <u>*isolated*</u> members subjected to a dynamic lateral impact.

Despite the difference in the level of the displacement, and the appearance of the small fluctuations, the responses for the two <u>*isolated*</u> members (with and without local effects) have similar shapes and frequencies (*Figure 4*). This indicates that the primary vibration in both is governed by the bowing mode of vibration in the member. The oscillations of the structural system take place about a displaced configuration. This similarity also exists between the responses of the two <u>*in-situ*</u> members.

It can be deduced from *Figure 4* that the predominate frequencies, the bowing frequencies, in the response of the <u>*in-situ*</u> members are lower than the governing frequencies from the corresponding <u>*isolated*</u> member models. Differences between the end conditions of the chord

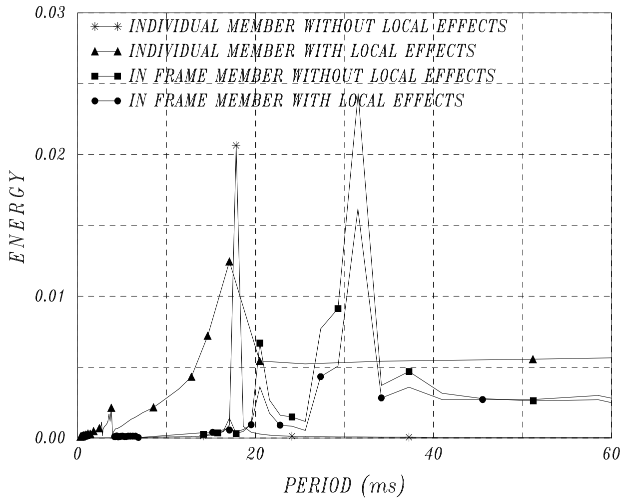

Figure 5 Period extraction of the responses displayed in *Figure 4* using Fourier transformation.

member in the frame compared with those used for the *isolated* member models seem to be the main source of the difference between the predominate frequencies. As mentioned in Section 3.1, the end conditions for the *isolated* member models are fixed to prevent rotation but allow translation along the axis of the member.

Figure 5 shows the periods of the responses of *Figure 4*, extracted using a Fourier transformation. The ordinate shows the energy spectra corresponding to each period and the abscissa shows the periods. The periods associated with the peak energies show the predominant periods of the vibrations for a stable response presented in *Figure 4*.

The dominant periods of the main frame models as well as the *isolated* members, with and without local effects included, are given in *Table 1*. The values shown in this table are obtained from *Figure 5*. The theoretical first natural periods of vibration for *isolated* tubular members with pinned and rigid ends are also given in *Table 1*. The Equations used for the calculation of the theoretical first natural period for vibration of pinned and encastred *isolated* tubular members are respectively (Clough and Penzien[5]) :

$$T = \frac{2}{\pi}\sqrt{\frac{\bar{m}L^4}{EI}} \tag{1}$$

$$T = \frac{8}{9\pi}\sqrt{\frac{\bar{m}L^4}{EI}} \tag{2}$$

It can be seen from *Table 1*, *Figure 4* and *Figure 5* that the dominant period of vibration of *isolated* members with and without local effects are close to the theoretical natural period of the corresponding tubular member with rigid ends. This shows the agreement between the numerical and the theoretical results.

Table 1 The dominant periods of vibration of the four main numerical models within a stable response.

Description	**Period of vibration (ms)**
Isolated member with no local effects	17.80
Isolated member with local effects	17.05
In-situ member with no local effects	31.50
In-situ member with local effects	31.50
Rigid end tubular member (theoretical)	16.74
Pinned end tubular member (theoretical)	37.96

With *in-situ* members, the predominant periods of vibration are almost twice those for the *isolated* members. The *in-situ* periods are much closer to the theoretical period of a member with pinned ends. This shows that under a dynamic impact, the end conditions for the chord member in the frame behave like pin connections. In Section 3.3 it was noted that for a quasi-static impact the end conditions for the chord member in the frame followed closely the response obtained from rigid end conditions.

Table 1, *Figure 4* and *Figure 5* show that the periods of an *isolated* member model, where local effects have been excluded, are less than the model where local effects have been considered (the frequencies are higher). This unexpected stiffer behaviour could be due to the fact that the dent produced in the impacted members is not sympathetic to the member bowing mode shapes and therefore the dented members become slightly stiffer for vibrations in the bowing mode. With the *in-situ* member, local damage again produces a slightly higher stiffness in the impacted member compared to that for the models where local effects have been ignored.

4.2 Unstable Responses

The oscillations presented in *Figure 4* are bounded and therefore the structural systems remain stable. By increasing the lateral impact load (or the pre-loading), the response of the structural system starts to become unbounded. In each of the four main numerical models, beyond a defined level of impact load, the responses become unbounded. This indicates that at this load level a dynamic instability has been propagated in the structural system. This dynamic limit point load, or the load which results in instability of the structure, is different in each model. For a MDOF (Multi Degree Of Freedom) system, an exact solution for the dynamic limit point load does not exist. Only Minimum Guaranteed Critical Loads (MGCL) can be evaluated (Simitses[14], Zeinoddini et al.[20, 27], Zeinoddini and Parke[25]).

Figures 6 and 7 show the response of two of the four main numerical models when different dynamic step lateral loads are applied at the mid-span of the chord member. Time histories of lateral deformation of the impacted position at mid-span of the damaged chord member are displayed in these figures. All the models carry a pre-loading which produces an axial compressive force equal to 50% of the chord member squash load (P_y). The stable and unstable responses can be distinguished in each figure. It can be seen that beyond a certain level of impact load, the responses become unbounded. This indicates that at this load a dynamic instability has been propagated in the structural system. This dynamic limit point load or, the load which results in instability of the structure, is different in each of the four models.

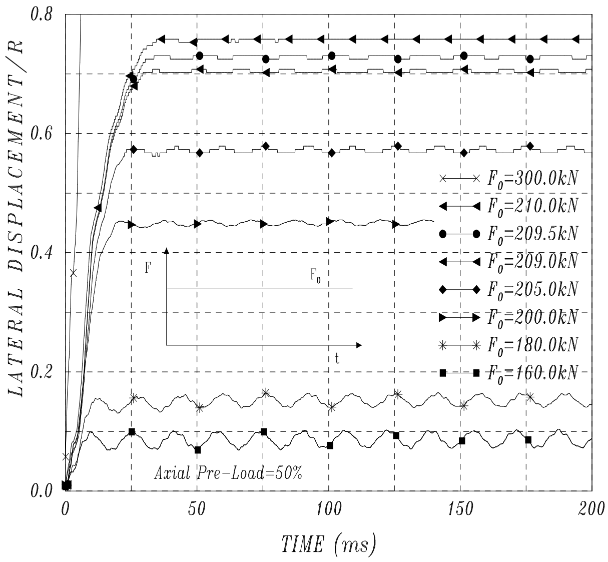

Figure 6 Time history of lateral deformation of the tubular member subjected to a dynamic lateral impact, where local effects have been included.

5 COMPARING THE QUASI-STATIC AND DYNAMIC RESPONSES

The quasi-static and dynamic behaviour of *isolated* impacted tubes have been addressed by Zeinoddini *et al.* [23, 24, 26] and Al-Thairy and Wang [2]. In this Section some differences observed between quasi-static and dynamic responses of *in-situ* impacted tubular members are discussed.

5.1 Failure Loads and Displacements

In *Tables 2* to *4*, the quasi-static and dynamic deformations, failure loads and failure displacements of the four basic numerical models used in the current study are compared with each other.

Table 2 compares the quasi-static and maximum dynamic lateral displacements under an impact load of 160 kN (or 0.22 $(8tD^2\sigma_y/L)$) which produces stable responses in all four models. The dynamic responses of the four models to this level of impact load have been shown in *Figure 4*. The displacements given in *Table 2* are dimensionless lateral deformations of the front elevation of the damaged chord member at the position of the impact load. For instance, in a frame model with local effects included the lateral displacement given in *Table 2* consists of the summation of denting, bowing and global deformations at the position of the

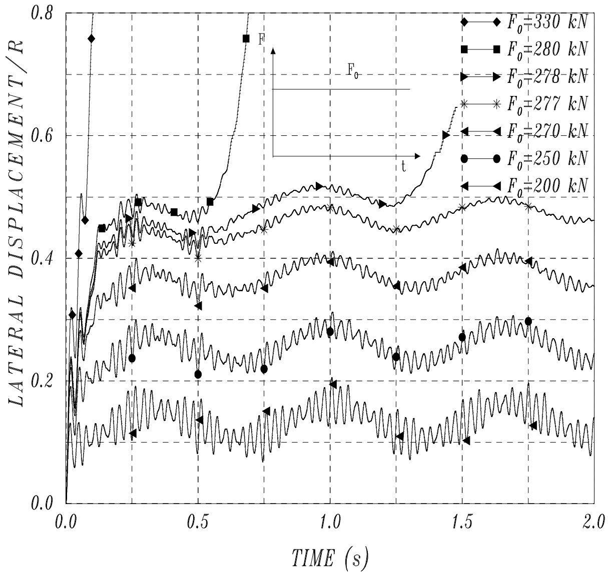

Figure 7 Time history of lateral deformation of the <u>*in-situ*</u> tubular member subjected to a dynamic lateral impact, where local effects have not been included.

impact load.

Table 2 Quasi-static and maximum dynamic lateral displacements of the four numerical models under stable impact loading (F_0=0.22 ($8tD^2\sigma_y/L$) or 160kN).

Description	**Static displacement /R**	**Dynamic (Max) displacement /R**	**Dynamic Amplification FActor (DAF)**
<u>*Isolated*</u> member with local effects omitted	0.0229 (1.00)	0.0579 (2.73)	2.53 (1.00)
<u>*Isolated*</u> member with local effects included	0.0298 (1.30)	0.1039 (4.54)	3.49 (1.38)
<u>*In-situ*</u> member with local effects omitted	0.0590 (2.58)	0.1180 (5.15)	2.00 (0.79)
<u>*In-situ*</u> member with local effects included	0.0705 (3.08)	0.2062 (9.00)	2.92 (1.15)

To ease comparisons, the displacements have been normalised using the quasi-static response of the <u>*isolated*</u> member with local effects excluded, the normalised figures being given in brackets. It can be seen from *Table 2* that the Dynamic Amplification Factor (the ratio between the maximum dynamic deformation and the corresponding static deformation) in the frame models is smaller than the similar values for the <u>*isolated*</u> member models. This is because the frame models have a relatively higher stiffness than the chord member. The Dynamic Amplification Factor has an inverse relationship to the stiffness of the system. Dynamic Amplification Factors are higher when local effects are included. This difference is due to the lower stiffness of the tube wall against lateral loading compared to the bowing stiffness of the member.

Tables 3 and *4* give the quasi-static and dynamic failure loads and failure displacements of the four main numerical models, respectively. The failure loads are the maximum lateral loads that can be resisted by the models. The failure displacements are the corresponding lateral displacements at the mid-span of the impacted member.

Table 3 Quasi-static and maximum dynamic lateral failure displacements of the four numerical models under extreme impact loading.

Description	**Quasi-static failure displacement/R**	**Dynamic failure displacement/R**	**Dynamic failure displacement/quasi-static displacement**
Isolated member with local effects omitted	0.112 (1.00)	0.344 (3.07)	3.07 (1.00)
Isolated member with local effects included	0.618 (5.52)	0.758 (6.77)	1.23 (0.40)
In-situ member with local effects omitted	0.316 (2.82)	0.443 (3.96)	1.40 (0.46)
In-situ member with local effects included	0.725 (6.47)	0.871 (7.78)	1.20 (0.39)

Table 4 Quasi-static and maximum dynamic lateral failure loads of the four numerical models under extreme impact loading.

Description	**Static failure load**$(8tD^2\sigma_y/L)$	**Dynamic failure load**$(8tD^2\sigma_y/L)$	**Dynamic failure load/ quasi-static failure load**
Isolated member with local effects omitted	0.619 (1.00)	0.393 (0.63)	0.63 (1.00)
Isolated member with local effects included	0.415 (0.67)	0.284 (0.46)	0.68 (1.08)
In-situ member with local effects omitted	0.617 (1.00)	0.385 (0.62)	0.62 (0.98)
In-situ member with local effects included	0.373 (0.60)	0.259 (0.42)	0.69 (1.09)

With a quasi-static impact, a maximum load value can be obtained and the corresponding displacement can be calculated (see *Figure 3*). With a dynamic impact load, no exact failure load can be defined. The dynamic failure loads given in *Table 4* correspond to the oscillation immediately below the Minimum Guaranteed Critical Loads. These load values appear to be close to the exact dynamic failure load. The dynamic failure displacements presented in *Table 3* correspond to the load values mentioned above. These may not be close to collapse values because of the potential rapid change in this critical region.

5.2 Circumferential and Longitudinal Deformations

Figure 8 compares the dynamic circumferential deformations of the *in-situ* impacted member, where local effects are included, with the corresponding quasi-static deformations. The deformed shapes in *Figure 8* show the cross-sections of the damaged tube at its mid-span. It can be seen that, with similar levels of denting, no significant difference can be observed between the shapes of dynamic and static circumferential deformations. Some researchers believe that the dynamic lateral loading of tubular members creates greater local deformations in the tube compared with a static lateral load (Søreide and Kavlie[15]; and Stronge[17]). From *Figure 8* it does not seem that the dynamic effects themselves have produced more local deformations in the circumferential direction.

The dynamic impacts did not show any significant differences to the deformations compared to the quasi-static impacts. This is because only small values of inertia forces are acting on the tube wall. The mass of the tube wall is insignificant compared to the mass of the frame and especially to the top-side mass. Consequently, the effect of the inertia forces in the tube wall, which can cause different shapes of cross-section in dynamic loading, becomes negligible. Therefore the dynamic effects themselves cannot cause greater local effects than the quasi-static impacts. A collision produced by an external mass can produce much larger inertia forces adjacent to the tube wall of the impacted member. In this case greater differences would be expected between quasi-static and dynamic deformations.

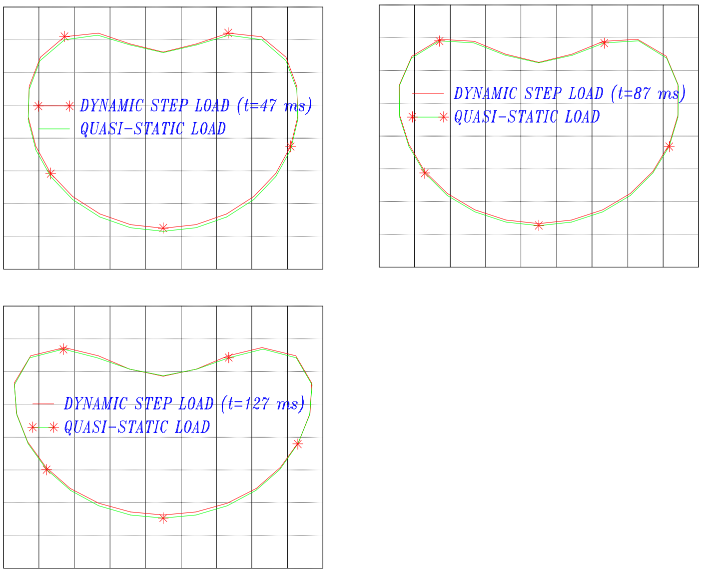

Figure 8 Deformed cross-sections of the *in-situ* chord member subjected to both quasi-static and dynamic lateral impacts.

Longitudinal deformations created by corresponding quasi-static and dynamic impact loads are compared in *Figure 9*. This figure shows the deformations of the front elevation of the tube along the impacted chord. In the abscissa the unity value represents the top of the chord member. Prior to a member buckling in the frame, the corresponding quasi- static and dynamic curves shown in *Figure 9* are close to each other with the dynamic deformations being very slightly higher. Again dynamic impacts have not caused more local deformation compared with the corresponding quasi-static impact.

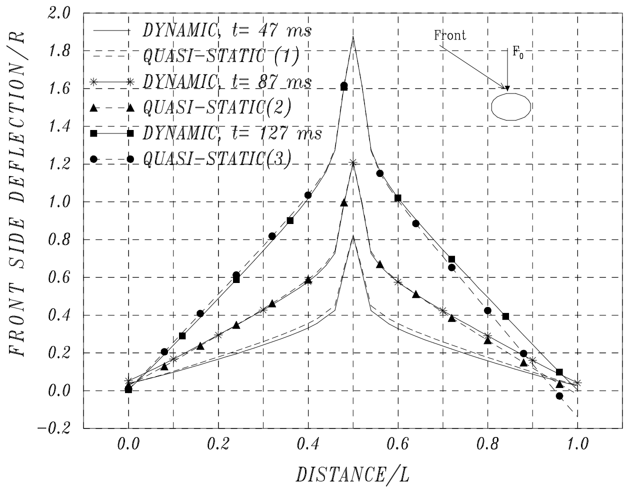

Figure 9 Deformation in the front elevation of the *in-situ* chord member subjected to quasi-static and dynamic lateral impacts.

5.3 Failure Modes

In *Figure 9* after member buckling occurring at t=106ms (between curves corresponding to t=87 and 127ms) there is a difference between quasi-static and dynamic deformations in the upper part of the impacted member. With a dynamic impact load, the top surface of the impacted member moves in the impact direction, while in contrast, under a quasi-static impact, the top surface of the impacted member moves in the opposite direction to that of the impact load. The displacement actually becomes negative for the quasi-static impact.

This difference reflects the occurrence of different failure modes in the tubular frame under these two types of impact loads. Under a dynamic impact load, failure occurs first in the upper bay compressive brace of the frame (see *Figure 10*). After buckling of this brace (with a reduction in the lateral resistance of the frame) there is an increase in the movement of the top end of the impacted member in the direction of the impact load.

Under quasi-static loading, in contrast to the dynamic impact, failure first takes place in the impacted member rather than in the bracing member. After failure, the member shortens and the vertical pre-loading produces a rotation of the top of the frame. As a result, the impacted member moves in the opposite direction to that of the impact load.

Most previous studies on ship-offshore collisions have concentrated on damage caused in the impacted member. The current study shows that the consequences of a collision may not be limited only to the impacted member and may extend to the connections or adjacent members.

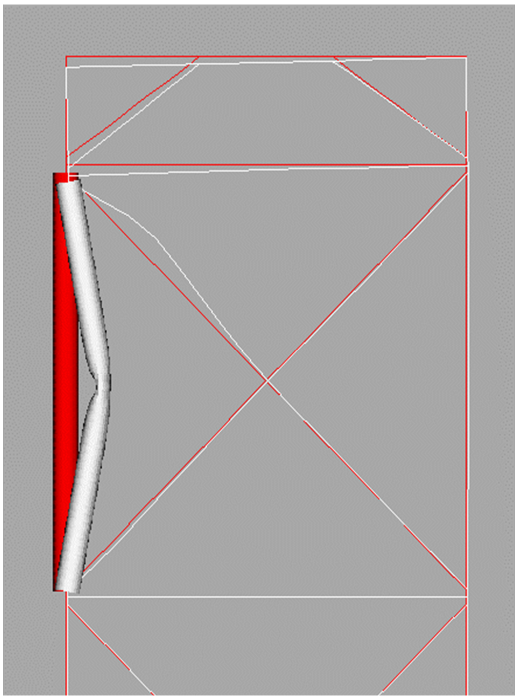

Figure 10 Failure modes in the tubular frame, subjected to a dynamic lateral impact (a brace buckles first).

In both frame models studied, with and without local deformations included, first failure was observed to occur in the impacted member if the impact was quasi-static. Failure occurred in a brace if the impact was dynamic. Dynamic amplification of the response is thought to have caused this important difference. When a stable structural system is dynamically loaded, the system oscillates about a deformed position that can be produced by the corresponding equivalent static loads. The maximum dynamic deformation will be bigger than the maximum static deformation achieved with the equivalent static load. With dynamic impacts, the global lateral displacement of the frame was amplified by the dynamic effects and exceeded a critical limit of lateral displacement. At this time the deformations in the impacted member were still far from the critical deformations the member could tolerate. This resulted in a failure in the bracing member prior to the failure in the impacted member.

Figures 10 and 11 show the deformed shape of the tubular frame, subjected to lateral dynamic and lateral quasi-static impacted loads. The deformed shapes have similar lateral displacements at the position of the lateral load. These figures show the structure after the occurrence of first member buckling. The onset of member buckling in a compressive brace in the upper bay can be clearly seen in the deformed shape of the frame subjected to the dynamic lateral impact (*Figure 10*). With quasi-static loading there is more axial shortening in the impacted member although this difference is not clearly visible in the figure.

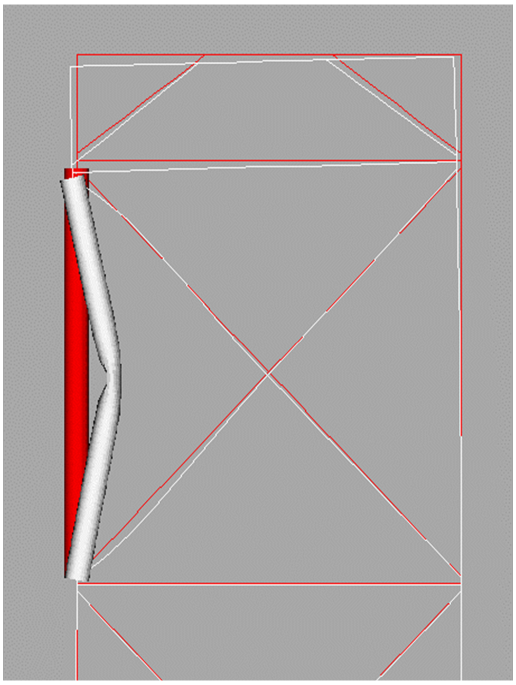

Figure 11 Failure modes in the tubular frame, subjected to a quasi-static lateral impact (the chord member buckles first).

6 CONCLUDING REMARKS

The response of an *in-situ* member in a structural frame subject to a lateral impact load is not necessarily the same as the response of an *isolated* impacted member. In this paper the behaviour of a chord member forming part of a tubular frame subjected to impact loads has been investigated. The tubular frame was tested experimentally by other researchers and reported in the literature. Non-linear numerical models of the frame have been validated using the experimental test results. These validated models have been examined under both quasi-static and dynamic impact loads with operational pre-loading applied. Some of the main differences between the response of a laterally impacted *in-situ* member and a corresponding *isolated* member have been reported in this paper.

With dynamic loading the impacted *in-situ* member has been found to behave rather similarly to that expected from an *isolated* member with pinned end conditions. With a quasi-static impact, the impacted in-situ member follows fairly closely the response obtained for a fixed ended isolated member. The end rigidity of the tubular member has a direct influence on the effective length of the member, which can subsequently affect the axial design load carrying capacity.

In the current study no significant difference has been found between the dynamic and quasi-static circumferential and longitudinal deformations, although some researchers believe that dynamic impact loads produce more local deformations in tubular members compared with a quasi-static load.

Whether or not local deformations have been included, first failure in the frame was observed to occur in the impacted member, if the impact was quasi-static. Failure occurred in a brace when the impact was dynamic. Most previous studies of ship-offshore collisions have concentrated on damage caused to the impacted member. The current study shows that the consequences of a collision may not be limited only to the impacted member but can extend to the connections and/or adjacent members. Some potential modes of failure might be overlooked if a quasi-static method of analysis is selected.

It should be emphasised that the results presented in this paper do not give a complete picture of the *in-situ* behaviour of laterally impacted steel tubular members. They are, however, able to illustrate some of the key factors involved. The tubular frame used in this study was a two bay, two dimensional frame. Real structures are more complex than this two dimensional benchmark frame. However, a number of important phenomena have been identified which will assist in future collapse analyses of complex offshore structures using non-linear software to determine the ultimate, and residual strengths of platforms.

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