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Blast Resistant Design in Cold Regions

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Abstract

An increasing interest of oil and gas production industries into the Arctic region has encouraged the research on cold resistant materials and structures. Since these oil and gas facilities handle hydrocarbons and other inflammable fuels, there have been incidences and always been a high potential risk of accidental explosions. In the past few decades considerable emphasis has been given to the safety concerns related to blast and achieving a 'Blast Resistant Design'. This paper provides an overview of the blast analysis techniques being currently employed in the industry to assess the dynamic response of structures subjected blast loading. In addition, a comprehensive overview is presented for the low temperature effects on the engineering properties of materials that are in use or that can be potentially used for the Arctic industrial structures. Currently, there are no specific industry standards or guidelines addressing blast resistant design of structures under low temperature conditions. Owing to this fact, this paper introduces various recommendations for blast assessments of both the existing and proposed structures in the Arctic or low service temperatures.

Keywords: Blast; Cold Temperature, Blast Design, Arctic

Soğuk Bölgelerde Patlamaya Göre Tasarım

Özet

Petrol ve gaz üretim endüstrilerinin Kuzey Kutbu bölgesine artan ilgisi, soğuğa dayanıklı malzeme ve yapılar üzerine araştırmaları teşvik etmiştir. Bu petrol ve gaz tesisleri, hidrokarbonları ve diğer yanıcı yakıtları kullandığından, kazalar meydana gelmiş ve her zaman yüksek potansiyel kazara patlama riski mevcuttur. Son yıllarda, patlama ve bir "Patlamaya Dayanıklı Tasarım" elde etme ile ilgili güvenlik endişelerine önemli ölçüde vurgu yapılmıştır.

Bu makale, patlama yüküne maruz kalan yapıların dinamik tepkisini değerlendirmek için şu anda endüstride kullanılan patlama analizi tekniklerine genel bir bakış sunmaktadır. Ek olarak, kullanımda olan veya Arctic endüstriyel yapıları için potansiyel olarak kullanılabilecek malzemelerin mühendislik özellikleri üzerindeki düşük sıcaklık etkileri için kapsamlı bir genel bakış sunulmaktadır. Şu anda, düşük sıcaklık koşullarında yapıların patlamaya dayanıklı tasarımını ele alan belirli endüstri standartları veya yönergeleri bulunmamaktadır. Bu gerçeğe bağlı olarak, bu makale, Kuzey Kutbu'nda veya düşük hizmet sıcaklıklarında hem mevcut hem de önerilen yapıların patlama değerlendirmeleri için çeşitli öneriler sunmaktadır.

Anahtar Kelimeler: Patlama; Soğuk Hava, Patlamaya Göre Tasarım, Kuzey Kutbu

1. INTRODUCTION

Arctic refers to those places at which the average temperature of the warmest month of the year is less than 10˚C [1]. Figure 1 shows the circumpolar Arctic region countries, which includes the most significant oil reserves and location of the Arctic Circle. The red isothermal line borders the areas with the average temperature of the warmest month below 10˚C. Yellow line shows the Arctic Circle and the gray areas indicates the largest oil fields located in the Arctic region.

Figure 1. *The Arctic region [1]*

In the Arctic region, minimum expected ambient temperatures are well below -40°C. Thus, minimum design temperatures down to - 60°C must be accounted for the structural assessments. Figure 2 shows the extreme minimum temperatures in Alaska and United States where the engineering properties of the constituent materials and members may be compromised.

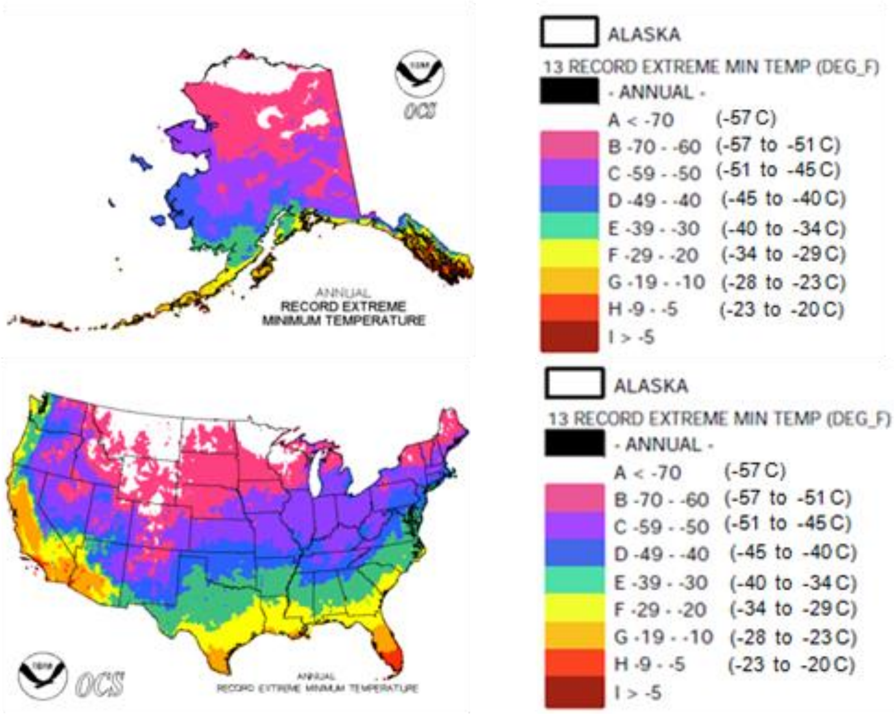

Figure 2. *Annual record of extreme minimum temperature in Alaska and United States [4]*

At such low temperatures, many engineering metals and alloys are transformed to brittle behavior so that structures fabricated from them fracture or shatter unexpectedly, when loaded to stress levels at which performance would be satisfactory at normal temperatures. Hence, it is very important to adopt appropriate service temperature for structural assessments. Estimates shall be comprised of the probability distributions of air temperatures that a structure is likely to encounter during its design service life. As per ISO 19902 [3], the lowest anticipated service temperature (LAST) shall be defined as minimum hourly average extreme-level (EL) air temperature. The extreme-level (EL) temperature is the temperature with an annual probability of exceedance not greater than 10^{-2}. Table 1 represents the lowest anticipated service temperatures for different locations.

Table 1. *Recommended lowest anticipated service temperatures (LAST) [5].*

LOCATION	**LAST in air**	**LAST in water**
Gulf of Mexico	− 10 °C (+ 14 °F)	+ 10 °C (+ 50 °F)
Southern California	0 °C (+ 32 °F)	+ 4 °C (+ 40 °F)
Cook Inlet, Alaska	− 29 °C (− 20 °F)	− 2 °C (+ 28 °F)
North Sea, south of Latitude 62°	− 10 °C (+ 14 °F)	+ 4 °C (+ 40 °F)
North Sea, north of Latitude 62°	Site-specific data should be used	
Mediterranean Sea, north of Latitude 38°	− 5 °C (+ 23 °F)	+ 5 °C (+ 41 °F)
Mediterranean Sea, south of Latitude 38°	0 °C (+ 32 °F)	+ 10 °C (+ 50 °F)

The oil and gas facilities handle hydrocarbons and other inflammable fuels that have a high potential risk of accidental explosions. Figure 3 shows few structures from the onshore and offshore petrochemical facilities in the Arctic that are typically susceptible to blast and explosion hazards. The consequence of blast explosion could not only cause catastrophic structural damage and financial losses but may also involve personnel and environmental (HSE) risks. Hence, achieving a Blast Resistant Design has become an important consideration for structural analysis from past few decades.

Figure 3. *Industrial structures in Arctic Region susceptible to blast*

The literary review has shown that, where blast resistant design often demands for a ductile behavior, most of the engineering metals exhibit an extremely contrasting brittle nature under low temperatures [2-3]. Majority of the prior research has focused on the independent behavior of material under blast loading and material behavior under cold temperature. This paper aims to augment the prior data and provide recommendations on key material response and design parameters. In addition, the recommendations or the modifications required in current design methodology to capture the combined effect of freezing temperatures and blast load on structural members is provided towards the end.

2.MATERIAL AND METHODS

2.1. Blast Analysis Methods

Assessing the dynamic response of structure against blast loading is a complex process. It involves the effect of dynamic deformation of the structure, high strain rates, and non-linear material behavior. The response to blast loads may either be determined by non-linear dynamic Finite Element (FE) analysis or by simple calculations models based on Single Degree of Freedom (SDOF) analogies and elastic-plastic methods of analysis. These methods are discussed briefly in the subsequent sections.

2.1.1. Equivalent static method

The dynamic load can have a significantly larger effect than a static load of the same magnitude due to the structure's inability to respond quickly to the loading (by deflecting). The increase in the effect of a dynamic load is given by the Dynamic Load Factor (DLF) or Dynamic Amplification Factor. Figure 4 shows the graphs of DLF vs. non-dimensional rise time (td/T = Duration / Natural period of the structure). The DLF is used to amplify the blast loads which is applied similar to wind loads and the structure is analyzed statically. This is a conservative approach and not vastly encouraged in the industry for detailed assessments.

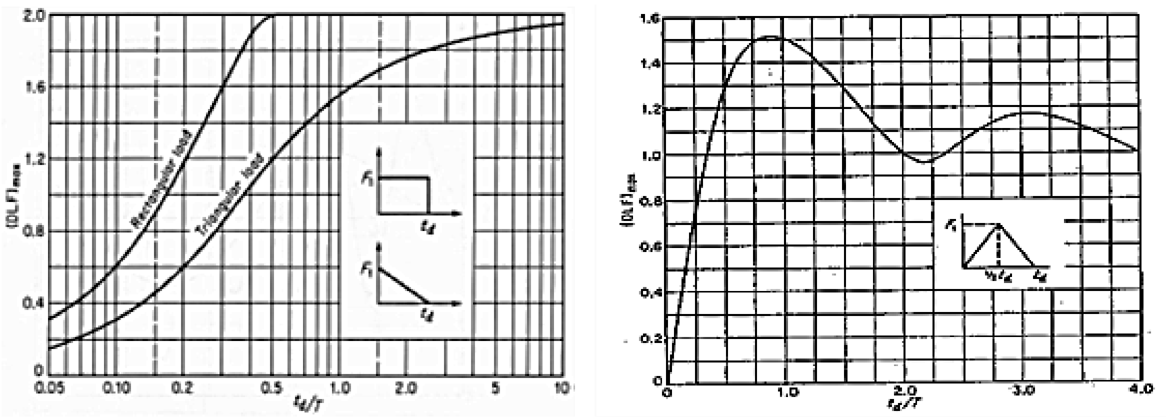

Figure 4. *DLF vs. non-dimensional rise time [9]*

2.1.2. Single degree of freedom (SDOF) analysis

In order to simplify the complexity of analysis, a blast analysis problem can be discretized into a simple SDOF analogy that is outlined in UFC 3-340-02 [9], Biggs text "Introduction to Structural Dynamics" [9], and ASCE's Design of Blast Resistant Buildings in Petrochemical Facilities [10]. These guidance documents use similar approaches in that the response of the structural system is modeled using an equivalent SDOF system with properties that are representative of the actual system. The SDOF model is developed so that in theory the same energy is required to displace both the analytical model and the real structural component. When performing a SDOF analysis, it is important to accurately develop the parameters that define the equivalent SDOF system, namely the mass, stiffness, and resistance. The ultimate resistance or strength of a component, and the stiffness are used to define a resistance-deflection relationship for a component. The ultimate resistance is typically expressed as the maximum uniform pressure that will cause a component to yield and is determined using conventional plastic design methods for hot-rolled steel but with higher material strengths. Blast loading is a short duration load also called impulsive loading. For the structural analyses the time history curve is assumed to be triangular with the peak pressure occurring instantly with zero rise time and decaying linearly till it reaches zero as illustrated in the left side of Figure 5. This assumption is conservative because it results in higher dynamic structural response. The ductility and natural period of vibration of a structure governs its response to an explosion.

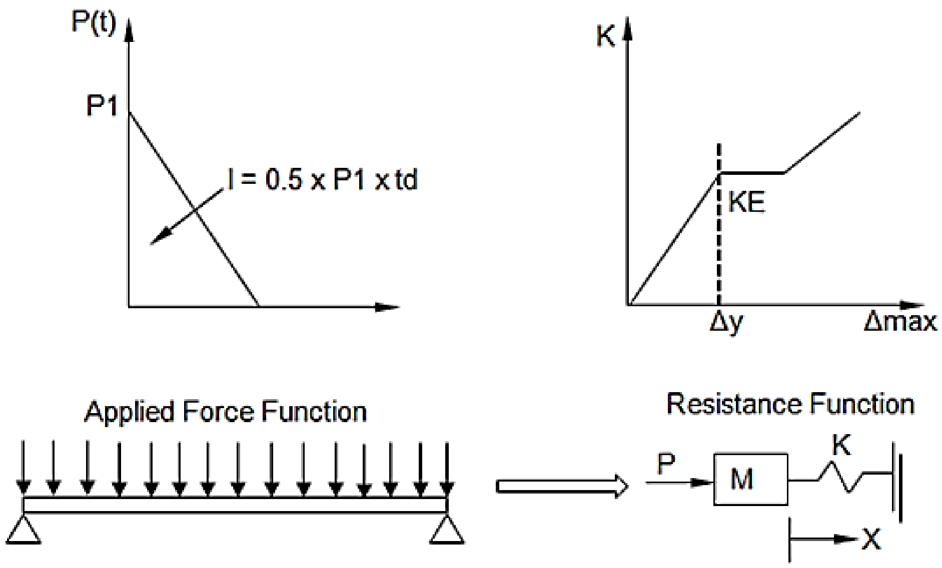

Figure 5. *SDOF system and blast loading [6]*

A typical resistance curve with tension membrane response is shown on the right side of Figure 5. The slope of the resistance-deflection curve in the elastic region is the elastic stiffness of the structural component. Tension membrane response occurs at relatively large deflections and when the boundary conditions prevent inward, in-plane movement of the supports so that catenary action occurs. The tension membrane response is often assumed to occur after flexural response. Compressive membrane response can also occur after flexural response in a component if it has rigid supports on both sides of the span length that do not allow rotation of the component cross section into the supports.

2.1.3. Finite element (FE) analysis

Finite Element (FE) Analysis is one of the robust approaches that are adopted in the industry, where structural complexity or spatial distribution of loading requires more detailed analyses. There are various commercial FE softwares which can represent complex geometries, connectivity, and material properties. These softwares relies on an explicit integration scheme which is suitable for performing dynamic, transient analysis of problems where large displacements occurs taking plasticity and failure into account. Capturing accurate structural response modes against blast and impact requires that correct, adequate and detailed information is included the model. While simplified models are prone to problems associated with too little information, complex models need to be tailored to available resources and time constraints. Figure 6 show few structures modeled using FEA software's and their response to blast loadings.

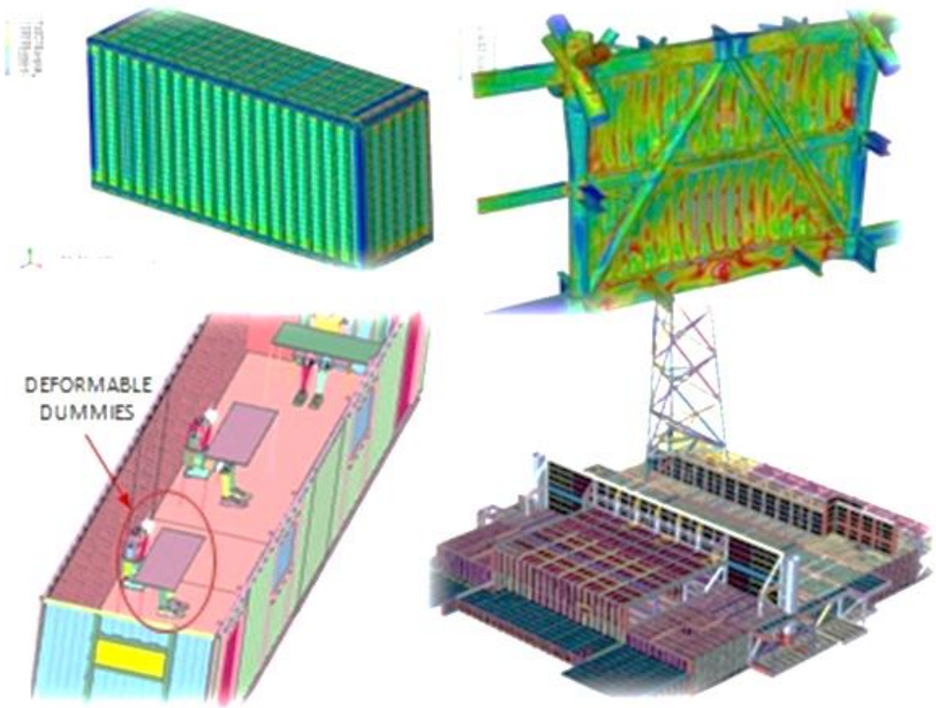

Figure 6. *FEA models showing blast response.*

3. GENERAL LITERATURE RESEARCH

3.1. Material Behavior Under Blast Loading

Structural materials are affected by extremely high loading rates, high pressures, and large inelastic deformations in response to blast loading. Hence, the material properties used in conventional design of structures are required to be modified when considering blast loadings to account for actual strength level and dynamic effects. The behavior of material under blast loading is discussed briefly in the subsequent sections.

3.1.1. Structural steel

Structural steel usually is stronger than the specified minimum strength. Therefore, for blast design the yield stresses should be multiplied by average static Strength Increase Factor (SIF). Also, steel mechanical properties vary with the time rate of strain. Structural steel is benefited from an increase in apparent strength when the rate of loading is rapid. The yield point increases substantially by a factor that is called the Dynamic Increase Factor (DIF) for yield stress. Table 2 and Table 3 show the strength increase factors for structural steel and aluminum respectively.

Table 2. *Strength increase factors for structural steel [10]*

MATERIAL	MINIMUM STATIC YIELD STRENGTH	(SIF)	(DIF)
Cold Formed Panels, Beams	200 – 400 MPa (30,000 - 60,000 psi)	1.21	1.10
Steel	200 – 240 MPa (30,000 - 36,000 psi)	1.10	1.29
Steel	290 – 400 MPa (42,000 - 60,000 psi)	1.05	1.19
Steel	515 – 690 MPa (75,000 - 100,000 psi)	1.00	1.09

Table 3. *Material strength increase factors for aluminum [10]*

TYPE OF ALUMINUM	YIELD STRENGTH	SIF	DIF
6061 T6	241 Mpa (35,000 psi)	1.07	1.02
6063 T5	110 Mpa (16,000 psi)	1.16	1.02
6063 T6	172 Mpa (25,000 psi)	1.12	1.02

3.1.2. Material modeling for stress-strain relationship

The stress-strain curves for the various steel grades can be approximated using conservative bilinear elastic-plastic material models as shown in Figure 7. In order to account for true plasticity in the non-linear (plastic) range, the stress-strain relationship can be converted from the typically given engineering stress-strain values into true stress-strain values. These true stress-strain curves can then be supplied as an input into Finite Element Analysis programs for blast analysis.

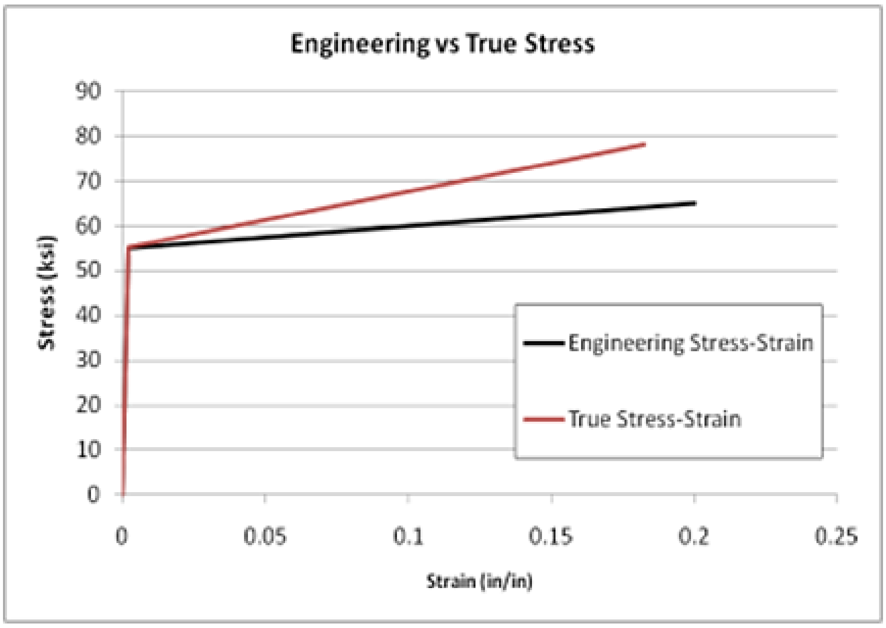

Figure 7. *Stress vs. strain curves*

3.1.3. Material modeling for strain rate effects

As discussed above, steel experiences increase in strength under rapidly applied loads (e.g. blast loads) as shown in Figure 8. The magnitude of the DIF depends on several factors including static material strength and strain rate.

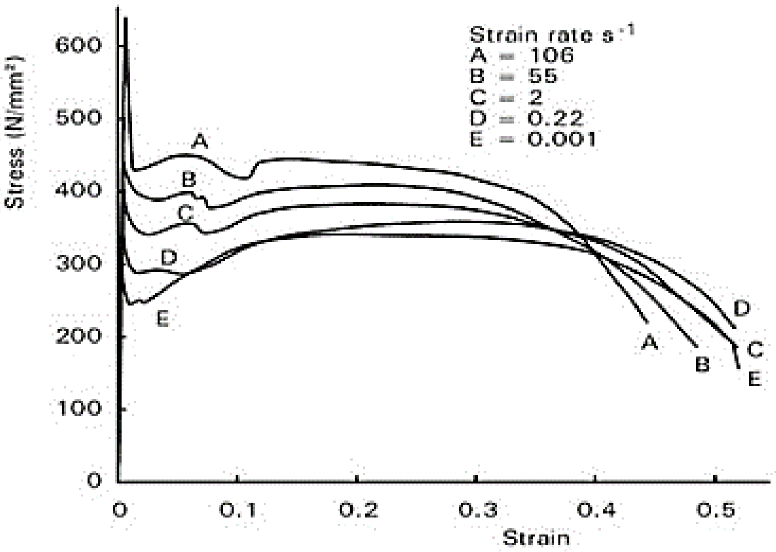

Figure 8. *Effects of strain rate on behavior of mild steel [11]*

The advantage of finite-element software is that it interpolates user supplied data and applies the proper strength increase based on the instantaneous strain rate encountered during a given increment. There is extensive research on dynamic strain rate effects on lower strength, mild steels and therefore well trusted empirical relationships are available. There are two well known equations for determining dynamic points on the stress-strain curve on low grade steels as discussed below. Cowper-Symonds [7] equation was derived from extensive tests on low strength steels.

$$\frac{\sigma_d}{\sigma_s} = 1 + \left(\frac{\dot{\varepsilon}}{D}\right)^{\frac{1}{q}} \tag{1}$$

Where, σ_d is the dynamic stress at a particular strain rate, σ_s is the static stress, is the uniaxial plastic strain rate. D and q are the constants which are specific to the steel. An alternative equation published by SCI [11] includes the specified yield stress as a variable.

$$\sigma = \sigma_t - 25 + 210.\left(\frac{\dot{\varepsilon}}{D}\right)^{\frac{1}{q}} \tag{2}$$

Where, σt is the specified minimum yield stress of the steel and D and q are constants for mild steel. The above relationship is given primarily to obtain data for steels with the yield stress in the vicinity of 355 Mpa (50ksi). The relationship however may not be valid for higher strength steels such as BS 7191: Grade 450 EM steels and BS EN 100025:S420 and S460 steels. Figure 9 shows that the SCI equation used in modeling 50ksi steel is conservative compared to Cowper-Symonds strain rate equation.

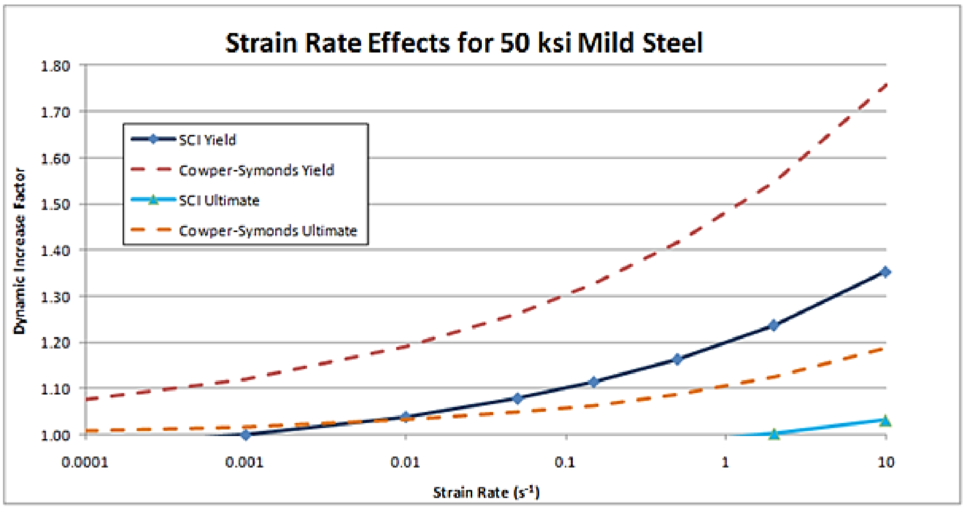

Figure 9. *Effects of strain rate on 50 ksi mild steel*

There are currently few tests for newer high strength steels used in offshore structures and no published relationships due to lack of research. One published test for a 450EMZ plate (Equivalent to API 2Y Grade 60ksi), shown in the graph below in the Figure 10.

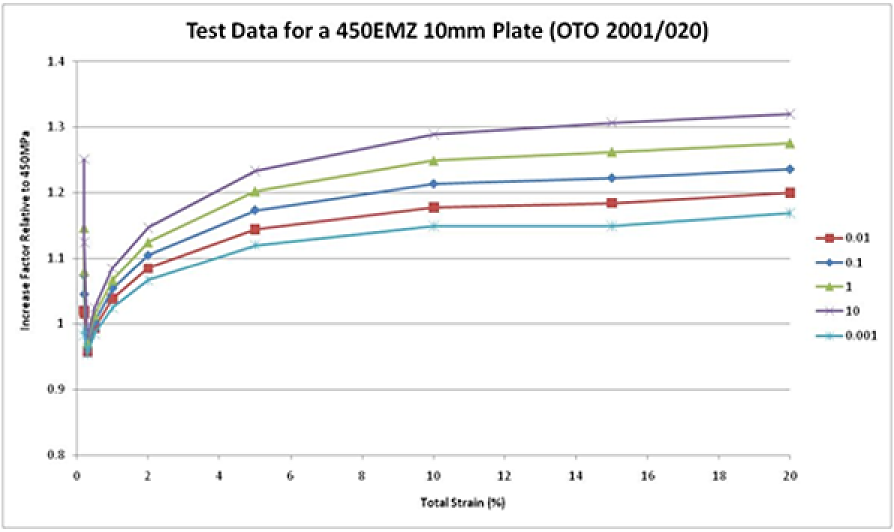

Figure 10. Effects of strain rate on grade 450 EM steel

3.1.4. Response criteria

Blast design and analysis are primarily component-based, whereby the applied blast load and response of each component in the building are determined. Performance goals are usually set in terms of life safety, functionality, and reusability for the entire building. Therefore, damage or response levels for individual building components must be established to achieve the overall building performance goal.

In SDOF or simplified MDOF (Multi Degree of Freedom) systems for blast analysis, the member failure is typically defined through two response parameters: Support rotation (Θ) and ductility ratio (μ). Ductility is the ratio between the maximum deflection and the maximum elastic deflection. Ductility less than unity denotes elastic behavior and greater than unity denotes the behavior is plastic. ASCE's Design of Blast Resistant Buildings in Petrochemical Facilities [6] has classified the deformation range in three different stages: low, medium, and high response as a function of the damage in the building. The response values for structural steel components are shown in Table 4 where, μ_a - Allowable Ductility Ratio and θ_a - Allowable Support Rotation.

Table 4. Response limits for structural steel components [10]

COMPONENT	LOW RESPONSE		MEDIUM RESPONSE		HIGH RESPONSE	
	μ_a	θ_a	μ_a	θ_a	μ_a	θ_a
Hot Rolled Steel Compact Secondary Members (Beams, Girts, Purlins)	3	2	10	6	20	12
Steel Primary Frame Members (with significant compression)	1.5	1	2	1.5	3	2

Steel Primary Frame Members (without significant compression)	1.5	1	3	2	6	4
Steel Plates	5	3	10	6	20	12
Open-Web Steel Joists	1	1	2	3	4	6
Cold-Formed Light Gage Steel Panels (with secured ends)	1.75	1.25	3	2	6	4
Cold-Formed Light Gage Steel Panels (with unsecured ends)	1	-	1.8	1.3	3	2
Cold-Formed Light Gage Steel Beams, Girts, Purlins and Non-Compact Secondary Hot Rolled Members	2	1.5	3	3	12	10

Similarly, the response values for few reinforced concrete (RC) components are shown in Table 5. Glass is often the weakest part of a building [8,12], breaking at low pressures compared to other components such as the floors, walls, or columns. Previous incidents have shown that glass breakage and associated injuries may extend many thousands of feet in large external explosions. High-velocity glass fragments have been shown to be a major contributor to injuries in such incidents.

Table 5. *Response limits for reinforced concrete components [10]*

COMPONENT	**LOW RESPONSE**		**MEDIUM RESPONSE**		**HIGH RESPONSE**	
	μ_a	θ_a	μ_a	θ_a	μ_a	θ_a
R/C Beams, Slabs, and Wall Panels		1		2		5
R/C Walls, Slabs, and Columns (in flexure, and axial compression load)		1		2		2

As part of the damage limiting approach, glass failure is not quantified in terms of whether breakage occurs or not, but rather by the hazard it causes to the occupants. Two failure modes that reduce the hazard posed by window glass are:

- glass that breaks but is retained by the frame
- glass fragments exit the frame and fall within 3 to 10 feet of the window

The glass performance condition is defined based on empirical data from explosive tests performed in a cubical space with a 10-foot dimension, refer Figure 11. The performance condition ranges from 1 which corresponds to not breaking to 5 which corresponds to hazardous flying debris at a distance of 10 feet from the window. Generally, a performance condition 3 or 4 is considered acceptable for buildings that are not at high risk of attack. At this level, the window breaks, fragments fly into the building but land harmlessly within 10 feet of the window or impact a witness panel 10 feet away no more than 2 feet above the floor level.

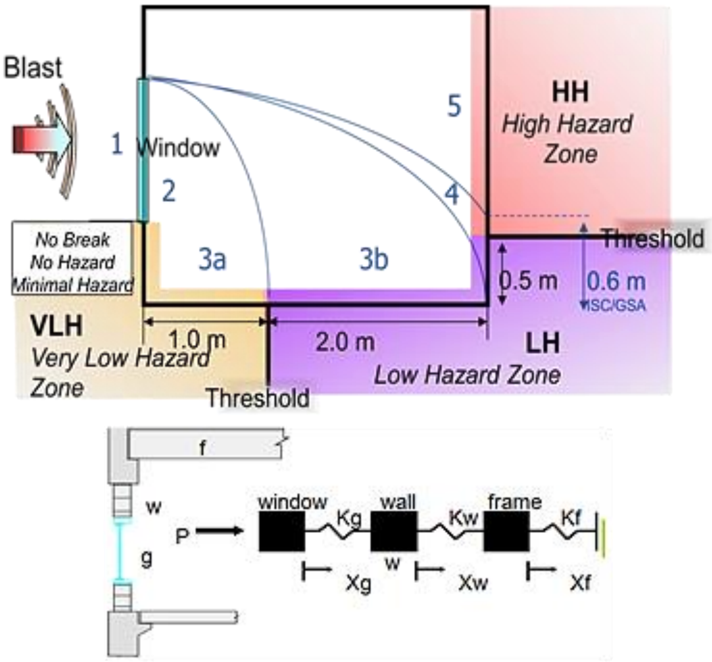

Figure 11. Window hazard classification

One of the most common solutions of mitigating the risk associated with glass is to apply anti-shatter film to glass so that the lamination holds shards of glass together in explosive events, reducing its potential to cause lethal injuries. The structural sealant helps to hold the pane in the frame for higher loads. Annealed glass is preferably used because it has a breaking strength is higher than the heat strengthened glass and tempered glass thus reducing the loads transmitted to the supporting frame and walls. The thickness of glass and film layer is selected based on the desired level of response and expected blast loading. If higher thickness of filming layer is used, the tension member forces into the framing members need to be considered in design.

As a representation of the typical test dummy response, Figure 12 presents a series of snapshots captured by the high-speed camera which used filmed glass. In the first view at earliest time, it was observe that the sheet of glass having already left the window frame and approaching the test dummy; the film is holding virtually all the fragments in a single sheet. Once the sheet impacts the head, some of the fragments become de-bonded from the film, particularly those which bounce back off the test dummy's head and back. As a result of the impact and the eccentricity of resistance applied to the sheet, the glass begins to rotate and eventually turns and passes over the test dummy's head.

(a) t=20 ms. (b) t=28 ms.

(c) t=34 ms. (d) t=49 ms.

Figure 12. *Response of test dummies exposed to blast impact of glass [12]*

3.2. Structural Steel at Low Temperatures

At low temperature the most significant property that changes in metals is the increase in brittleness and consequently the loss of ductility. Mechanical properties of steel that increase as temperatures decrease are the Yield Strength, Tensile Strength, Modulus of Elasticity, Strain rate. However, other important mechanical properties of steel that decreases in cold temperatures are the Ductility, Material Toughness or Impact Strength, Poisson's Ratio, Thermal Expansion Coefficient, Specific Heat. These properties are discussed in detail in the subsequent sections.

3.2.1. Material toughness

Material toughness can be defined as the ability to deform plastically in the presence of a flaw subjected to load rather than fracturing in presence of high stress concentrations. Toughness is commonly measured in terms of energy absorbed in breaking under impact loading [Charpy V-notch test]. The notch toughness of steel depends on the ambient temperature, material composition, heat treatment, loading rate, state of stress ahead of the notch. The state of stress ahead of the flaw will depend on tensile stress, flaw size and plate thickness. At a given test temperature, a metal may manifest a high ductility in the tensile test and practically no toughness in a notched bar impact test. Also, the fracture toughness can increase with decreasing plate thickness as a result of the relaxation of the lateral constraint in the vicinity of the notch tip Figure 13.

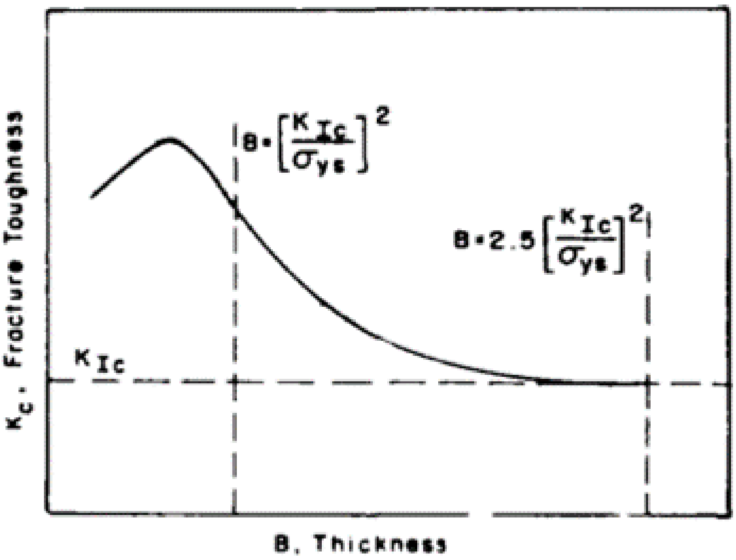

Figure 13. *Transition of Fracture Toughness [13]*

3.2.2. Transition temperature

The temperature at which the ductile property of steel is transformed to a brittle behavior is called as the transition temperature. This transition temperature at which brittle fracture occurs is lowered by a decrease in carbon content, velocity of deformation, depth of notch, grain size and increase in the radius of notch, increase in nickel and manganese content.

Figure 14 shows the energy-temperature curves obtained by charpy V notch tests of steels having different chemical compositions [13]. As discussed, the metal composition has significant influence on the transition temperatures. Past research has shown that low alloy and high nickel content steels has excellent properties in reducing the transition temperature and increasing ductility. However, the high nickel content steels are comparatively expensive and mostly used in the industry for cryogenic applications.

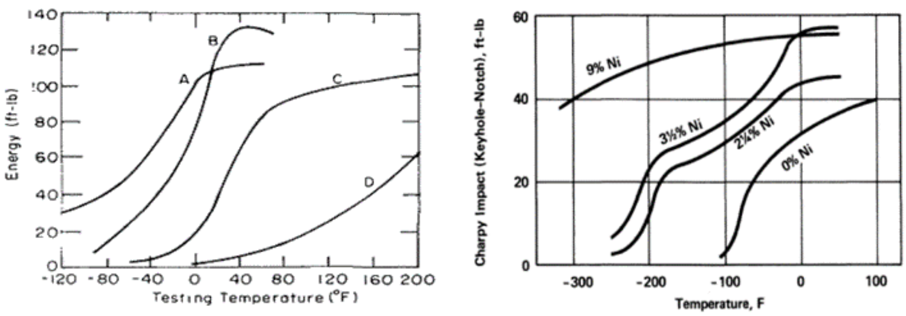

Figure 14. *Energy Temp Curves Obtained by Charpy V test [13]*

Figure 15 shows the toughness and temperature relationship obtained from charpy V notch test for carbon steel that is used for ordinary structures, where very low temperatures are not expected. The ductility transition temperature in the figure indicates that the material (here carbon steel) is expected to undergo brittle fracture when the service temperature is below 17°F (-8.33 °C).

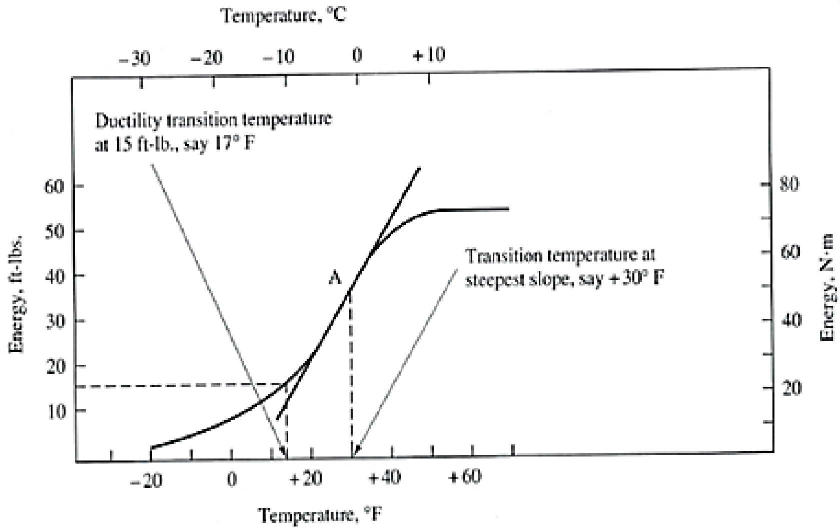

Figure 15. *Transition Temperature curve for Carbon Steel obtained from Charpy V Impact test [13]*

There is another concept of nil ductility transition (NDT) temperature which typically depends on the material thickness and rate of loading. This temperature indicates the highest temperature at which a standard specimen fails in a brittle manner under dynamic loading. At temperatures above the NDT temperature, the material has sufficient ductility to deform in-elastically before total fracture. Below the NDT temperature, the fracture toughness remains relatively constant with changing temperature. Thus, for structures subject to static or dynamic loading, the respective fracture toughness-to-temperature relations must be used to characterize the fracture behavior. For impact loading, the NDT temperature approximately defines the upper limit of the plane-strain condition.

Loss of ductility in a metal can be observed by examining its low temperature stress-strain relationship. As discussed above, with the decrease in temperature both the yield strength and the ultimate strength point may shift to a higher stress value, but fracture may begin at a much lower strain value. This phenomenon can be observed in Figure 16 which shows the strength vs. temperature plot for pure iron in simple tension test where at a certain temperature, the yield stress and fracture stress meet; and below this temperature, exhibiting a brittle behavior.

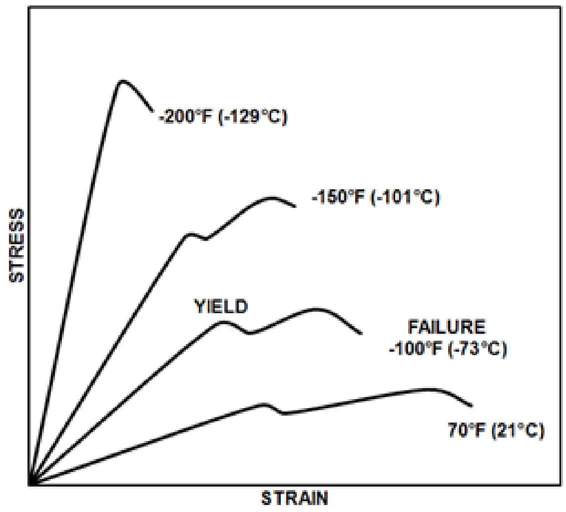

Figure 16. *Typical stress-strain curve of body centered cubic class metal at decreasing temperatures [13]*

3.2.3. Strength and elongation vs. temperature

Ductility is commonly expressed in terms of percentage elongation in gauge length, and reduction in area, of a tensile specimen that is tested to fracture. Figure 17 shows the analogy between low carbon steel and 9% Ni steel. At low temperatures an obvious increase in the yield and tensile strengths was observed in both types of steels. However, where low carbon steel shows a loss of ductility at low temperatures, the elongation of 9% Ni steel held the constant values throughout the temperature variation This indirectly implies that Ni steel has superior resistance to brittle fracture at low temperatures.

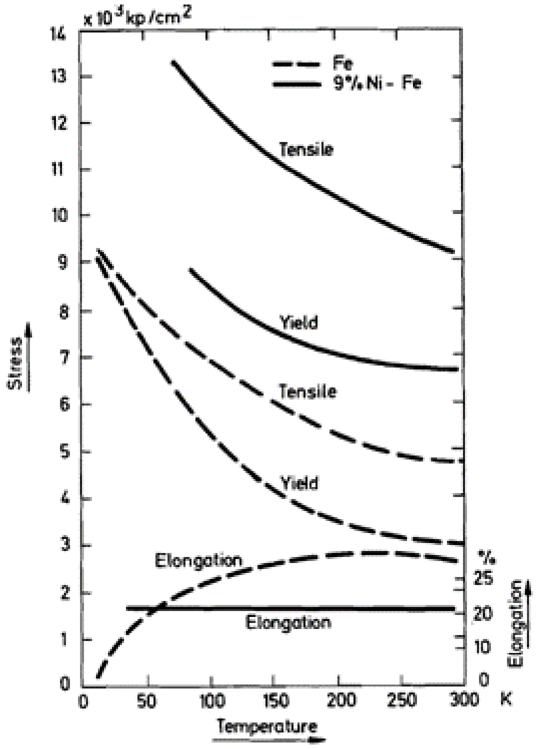

Figure 17. Ultimate tensile, yield, and elongation of low carbon and Ni (9%) – steel vs. temperature [15]

3.2.4. Effects of strain rate

Rapid or high strain-rate loading affects the mechanical properties of structural steels and the same are discussed in this section briefly with reference to low temperature conditions.

At low and normal temperatures, the typical effects of increased strain rate on the response of structural steels are an increase in yield stress; an increase in ultimate strength, though smaller than for yield stress. At elevated temperatures, increase in the strain rate has a relatively small influence on the yield strength, however, a slight decrease in the tensile strength of most of the steels was observed. As discussed above, ductility of structural steels, as measured by elongation or reduction of area tends to decrease with rapid increase in the stain rates. Typically, reduction of the elongation was observed with decreasing temperatures with increasing strain rates. Figure 18 shows another results of tensile test conducted on 9% Ni steel at room temperature and low temperature with varying strain rates. A typical strain rate hardening behavior of steel was observed at both temperatures. When compared with the case of 293 degrees K, as a whole, all of the curves showed a significant increase in strength. At a low strain rate state, it showed an obvious yield point on the curve and a smooth hardening behavior up to the strain of 20%. As the strain rate increased, however the behavior was changed slightly. There appeared a significant softening behavior after the stress reached peak value. At a strain rate of 0.33 s^{-1}, there was no strain hardening region, only significant softening subsequent to some plastic deformation after yielding. At a high strain rate of 2.5 x102 s^{-1}, the trend became more obvious and it showed a significant softening with increase of strain after the stress reached a peak value at a small strain.

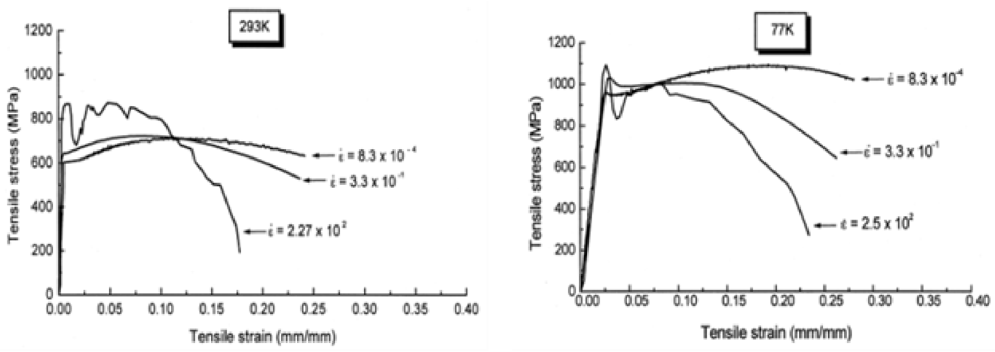

Figure 18. *Tensile stress against the tensile strains at room temperature (RT) and −196°C [16]*

Table 6. *Temperature and Strain Rate dependence on Yield Strength per Eurocode [17].*

STEEL GRADE	$f_y(\dot{\varepsilon},T)$	**m**
S235	$f_{y,RT}+960.\left[1-1.0767x10^{-4}.T.\ln\left(\frac{10^{8}}{\dot{\varepsilon}}\right)\right]^{m}$	2.80
S275		
S355		3.27
S460		
S690	$f_{y,RT}+960.\left[1-7.2993x10^{-5}.T.\ln\left(\frac{10^{10}}{\dot{\varepsilon}}\right)\right]^{m}$	3.74
S890		

3.2.5. Standard guidelines for choice of material for low service temperatures

ASTM [14] has specified the impact test requirements for different grades of structural steel, based on the intended loading behavior and expected service temperature. Similarly, those products which are intended to be used as tension components of fracture-critical members are expected to meet some of the requirements as shown in ASTM [18].

For the purposes of material selection and utilization in offshore structures, ISO [5] has characterized steel into different strength groups and toughness classes. The minimum toughness requirements based on different toughness classes. The weld metal and HAZ (Heat affected zone) charpy toughness requirements should be demonstrated during WPQ (welding procedure qualification) depending on material strength group and thickness.

Eurocode [17] has also given guidance for selection of materials for fracture toughness through thickness properties.

3.3. Reinforced Concrete at Low Temperatures

Research has shown that cold temperature causes a significant increase in the compressive strength, modulus of elasticity and stiffness of reinforced concrete members with a reduction in displacement - ductility capacity. Also a substantial increase has been observed in other properties such as the steel-concrete bond strength, concrete fracture energy and the concrete tensile strength. Figure 19 shows a considerable reduction in deformation of the concrete member at -40°C compared to when tested at 20°C. These properties under low temperatures are discussed briefly in the subsequent sections.

3.3.1. Compressive strength of concrete

Increase in the compressive strength of concrete (fc') at low temperature mainly depends on the moisture content and to a lesser extent on the characteristic of the mix e.g. the water cement ratio and the air content [19]. Figure 20 shows that saturated concrete at low temperature exhibit substantial increase in strength compared to partially dry concrete. It was observed that the increase in strength is due to the water expansion when transformed to ice. This is implied in Figure 20 which shows that the compressive strength of concrete increases with the increase in water cement ratio.

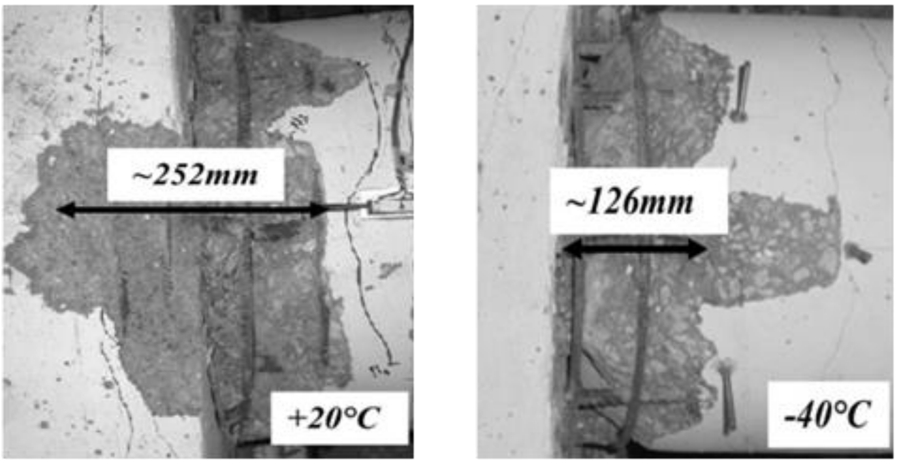

Figure 19. *Concrete behavior at low temperature [19]*

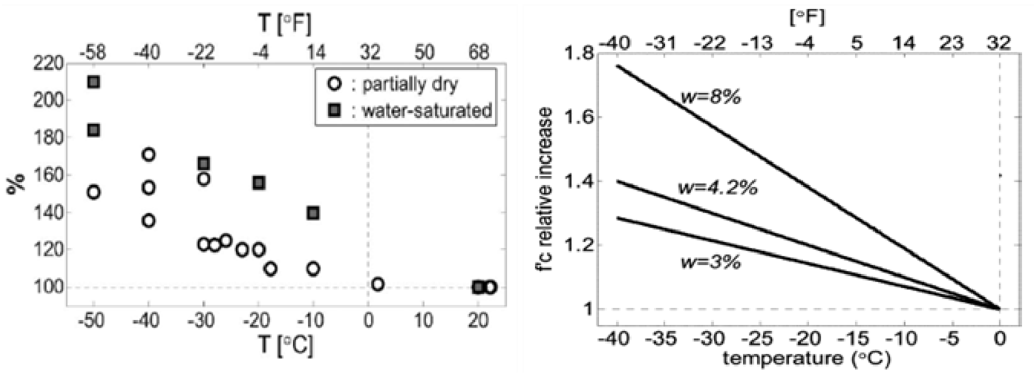

__Figure 20.__ Increase in compressive strength of concrete at low temperatures [19]

Based on these experimental observations, researchers have proposed empirical equations to determine the additional strength of concrete at low temperatures. Figure 21 shows the evaluation of these equations with the data collected from partially dry concrete having moisture content of 3% and compressive strength of 28MPa (4 ksi) at the room temperature.

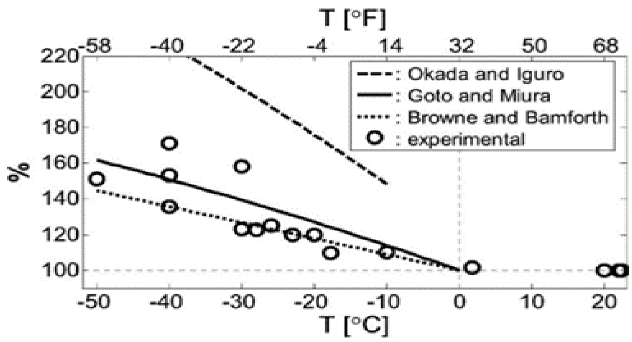

__Figure 21.__ Relative increase of concrete compressive strength at low temperatures [19]

3.3.2. Modulus of elasticity

Figure 22 shows that rate of increase of concrete modulus of elasticity, E_c at low temperatures is smaller than that for the compressive strength.

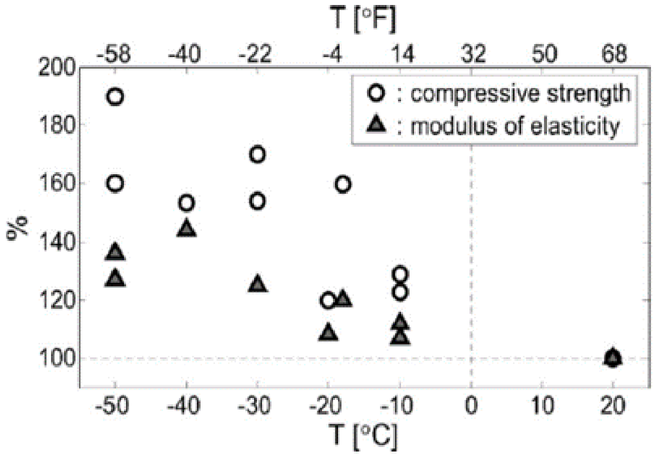

Figure 22. *Concrete modulus of elasticity at various temperatures (expressed as percentage of values measured at 20°C) [19]*

At normal temperature, the modulus of elasticity (E_c) of concrete is approximately proportional to its square root. The effects of low temperature on E_c can be considered by using ACI equations [18] which includes the temperature effect.

3.2.3. Reinforcing steel

Experimental research has shown that the yield stress and the tensile strength of the reinforcing steel enhances at the same rate with reducing temperatures. On the other hand, Figure 23 shows no significant effect on the deformation capacity of the reinforcing steel.

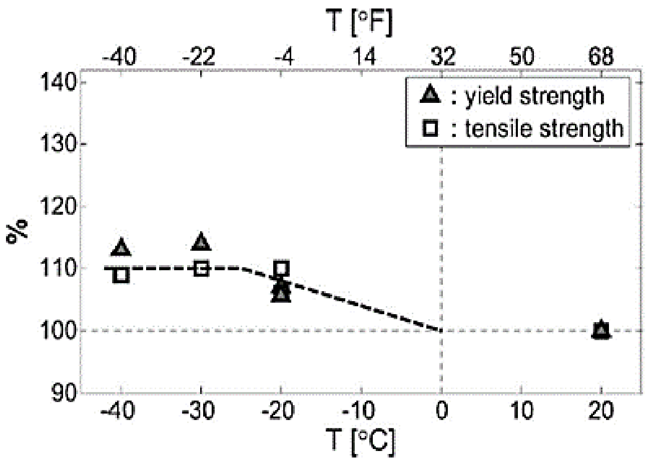

Figure 23. *Yield and Tensile Strengths of Steel Reinforcing Bars at Various Temperatures (Expressed as a Function of the Values Measured at 20°C) [19]*

3.2.4. Low temperature effects on the reinforced concrete (RC) columns

The columns that studied generally were divided into three groups according to their structural behavior: 1) flexural-dominated ordinary RC (ORC) columns; 2) flexural-dominated RC filled steel tube (RCFST) columns; and 3) shear dominated. Figure 24 shows the result obtained for ORC columns, the flexural-dominated columns tested at low temperatures (–40°C) exhibit an increase in the flexural strength and a reduction of around 20% in the displacement capacity. A significant increase in the modulus of elasticity was observed.

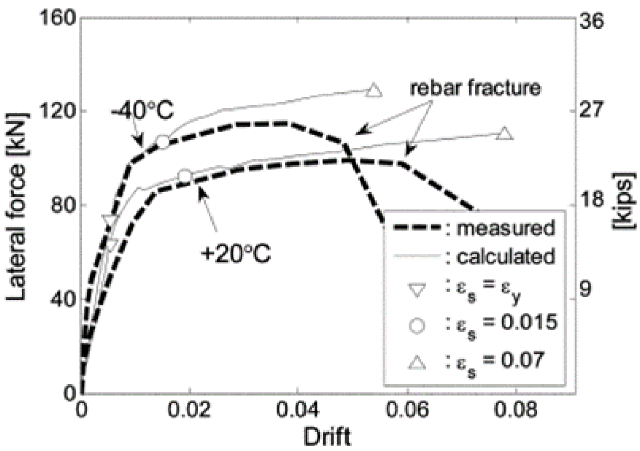

Figure 24. *Low Temperature Effect in Cyclic behavior of Flexural dominated columns [19]*

A parametric study was performed to quantify the increase of flexural strength at low temperatures. Figure 25 shows the over strengths of around 95 section configurations along with units tested above [19]. A low temperature flexural over strength of around 15% was observed for the RC columns.

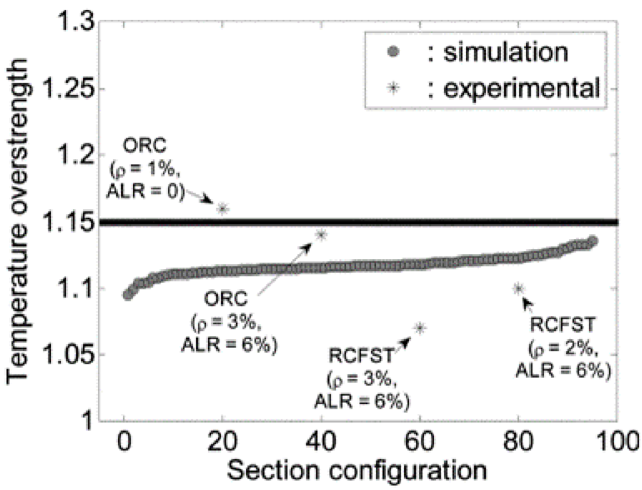

Figure 25. *Low Temperature Flexural over strength for RC Columns [19]*

3.2.5. Displacement-ductility capacity

Figure 26 and Figure 27 show the reduction of displacement ductility capacity as a function of concrete and steel strain, respectively. Up to 40% reduction in the ductility capacity as observed that at -40°C The empirical equations used to estimate the low temperature ductility capacity reduction for concrete 'ε_c' or steel 'ε_s' strain is indicated below the curves in the subsequent figures.

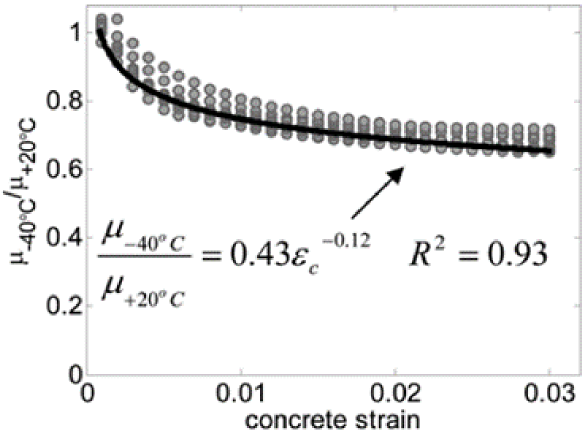

Figure 26. Reduced Ductility at low Temperatures as function of Concrete Strain [20]

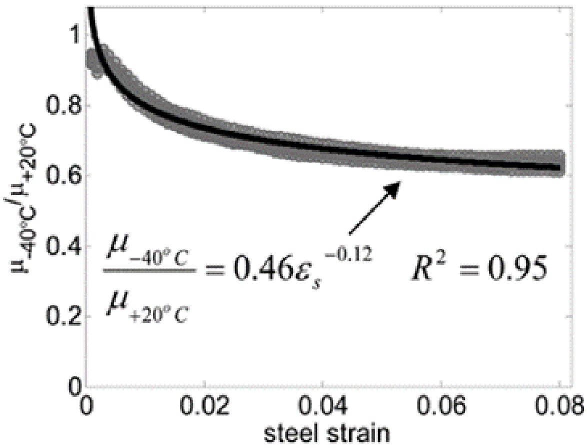

Figure 27. Reduced Ductility at low Temperatures as function of Steel Strain [20]

3.3. Glass at Low Temperature

The breaking stress depends upon the ambient condition and the amount abrasion. Table 13 refers to the test conducted on a glass at various temperatures.

Table 7. *Breaking stress of glass at low temperatures, Median values from probability plots [21]*

RATE STRESS INCREASE (MPa sec^{-1})	BREAKING STRESS (MPa)			
	296°K (23°C)	194°K (-79°C)	76°K (-197°C)	20°K (-250°C)
5.51	51.7	65.5	71.7	71.7
0.07	37.9	51.7	71.7	73.1
0.007	34.8	44.1	71.7	70.3

Following were the conclusions at the end of the test:

- Glass failure strength increases with the decrease in temperature.
- The breaking strength at -197 deg C and -250 deg C is nearly the same
- The modulus of elasticity changes by less than 2% over this temperature range investigated.
- Low temperatures glass exhibits very little or no fatigue, and therefore much higher design stresses can be used for glass.

4. CONCLUSION AND RECOMMENDATIONS

Since steels are susceptible to brittle fracture at low temperatures, material with sufficient fracture toughness at the temperature, strain rate etc. should be selected. Fracture control should be considered in all material selection and structural design activities. Steel materials with low ductile-to-brittle transition temperature of your steel (or, NDT - Nil Ductility Transition) should be selected. Using thinner sections (less than 50 mm) are better from a fracture toughness point of view (not exceeding required stress levels). The existence of multiple load fracture paths or redundancy so that a single fracture cannot lead to a complete failure of the structure. Differential strains or residual stresses due to temperature changes between construction and permanent locations shall be considered in the design (especially for Blast Resistant Portable Buildings).

Steel structures:

- Steel Structures should be design for only “Low response level”
- Sliding analysis should be performed for Blast Resistant Portable buildings to prevent occupant injuries
- The steel material should have:
    i. (Tensile Strength/Yield Strength) ≥ 1.2
    ii. Minimum elongation of 15%

• Finite Element Analysis should be preferred to identify stress concentrated areas. Maximum plastic strains of 5% and 0% are recommended in the structural members and at the connections (welds) respectively.

• An Over-strength factor of 1.5 is suggested to design welded connections

Reinforced Concrete Structures:

• An Over-strength factor of 1.2 is suggested to design the connections

Recommended Response Limits for Reinforced Concrete Components for Z2 and Z3 were given in Table 8 below.

Table 8. *Recommended Response Limits for Reinforced Concrete Components for Zones 2 and 3.*

COMPONENT	LOW RESPONSE		MEDIUM RESPONSE		HIGH RESPONSE	
	μ_a	θ_a	μ_a	θ_a	μ_a	θ_a
R/C Beams, Slabs, and Wall Panels	-	0.8	-	1.6	-	4
R/C Walls, Slabs, and Columns (in flexure, and axial compression load)	-	0.8	-	1.6	-	1.6

Where, μ_a - Allowable Ductility Ratio and θa - Allowable Support Rotation

Glazing System (Window Glass):

• Use material strength at room temperature to design/analyze blast resistant window glass

• An over-strength factor of 1.3 should be used to design window frame and other supporting members for blast design for Zone 2 and 3.

• Window glass should be designed for “NO BREAK/NO HAZARD” response level.

Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

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