Finite Elements Analysis Techniques of Vessel Collision with Cable-Stayed Bridge

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Abstract: Vessel collision design for bridges crossing navigable waterways is an important consideration since it significantly affects the total cost of bridges. Economical design requires appropriate determination of impact loads imparted to bridge piers. While the collision force is dynamic in nature, current provisions for bridge design are based on static approximations of structural behavior and limited experimental data, it prescribed by current bridge design specifications (Method II probabilistic approach as outlined in " the AASHTO Guide Specifications for Vessel Collision Design of Highway Bridges"). Collision force and structural deformations predicted by the static and the dynamic analysis techniques are compared for vessel collisions of varying mass (DWT) and Velocity.This research is concerned with the effect of vessel impact forces on long span cable-stayed bridge. The Contact-Stiffness Approach was applied to determine the maximum impact force of a vessel collision as a function of the vessel velocity, and the deadweight tonnage of the vessel. Impact force is applied to the tower of bridge at the point above water level. A comparative study was conducted to investigate the effect of vessel impact force on Tatara cable-stayed bridges, with a center span of 890 m, cases of loading with different values of the vessel velocity, and deadweight tonnage of the vessel were studied for Static and dynamic Analysis finite element bridge Structure using ANSYS program. Results from such comparisons indicate that, dynamic analysis technique are preferable. For more severe collision conditions, the use of equivalent static force for design purposes is acceptable.

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1. Introduction

Bridges crossing coastal or inland waterways are susceptible to collapse caused by vessels impacting bridge piers. The increase in vessel size and traffic density has put these bridges at higher risk of being hit *(Saul and Svensson,1983)* Direct inclusion of veseel -ship and barge- impact loads on bridge structures was neglected in bridge design until about twenty-five years ago. The possibility of such a catastrophic collision was considered very small and therefore disregarded. Additionally, designing bridges to resist such an extreme event could be overly conservative and uneconomical. Moreover, methods for determining impact forces were not well understood or established.

Consideration for the design of bridges against vessel impact is important in many countries around the world. Land-locked countries must be concerned with vessel traffic in rivers, channels and lakes, while countries by the ocean must account for vessel traffic entering and leaving its ports. Vessels have been known to collide with other vessels, with bridge piers, and with other obstacles. Countries like the United States, Japan, and Germany have, over the years, carried out numerous research studies dealing with vessel impact on bridges and other obstacles.

Vessel collisions on bridge structures may occur when vessels veer off-course, becoming aberrant. Factors that affect vessel aberrancy include adverse weather conditions, mechanical failures, and human error. It has been noted in the literature that, on average, at least one serious vessel collision occurs per year *(Larsen, 1993)*. During severe vessel collisions, significant lateral loads may be imparted to bridge structures. Engineers must therefore account for lateral vessel collision force when designing bridge structures over navigable waterways. If such bridges cannot adequately resist impact loading, vessel collisions may result in failure and collapse of the bridge; leading to expensive repairs, extensive traffic delays, and potentially, human casualties.

It was only after a marked increase in the frequency and severity of vessel collisions with bridges that studies of the vessel collision problem have been initiated in recent years. In the period from 1960 to 1998, there have been 30 major bridge collapses worldwide due to vessel collision, with a total loss of life of 321 people. The greatest loss of life occurred in 1983 when a passenger ship collided with a railroad bridge on the Volga River, Russia; 176 were killed when the aberrant vessel attempted to transit through a side span of the massive bridge. Most of the deaths occurred when a packed movie theater on the top deck of the passenger ship was sheared off by the low vertical clearance of the bridge superstructure.

Of the bridge catastrophes mentioned above, 15 have occurred in the United States, including the 1980 collapse of the Sunshine Skyway Bridge crossing Tampa Bay, Florida, in which 396 m of the main span collapsed and 35 lives were lost as a result of the collision by an empty 35,000 *DWT* bulk carrier, is shown in Fig. 1.

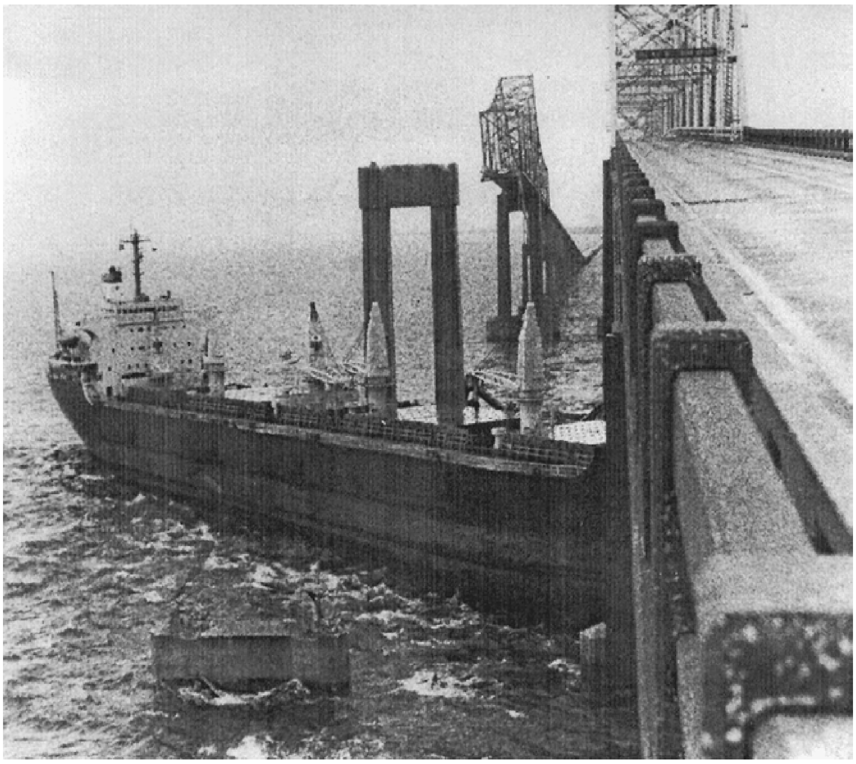

Fig. 1. The 1980 collapse of the Sunshine Skyway Bridge

2. Estimating Vessel collision force

In the design and evaluation of bridge structures that cross navigable waterways, the loads imparted to a bridge during potential vessel impact events must be carefully considered. The guidance on vessel impact forces can be classified according to the governing assumptions made to estimate the maximum forces. The three basic approaches are ***(Rober and Stevent, 2002)***[7]:

- **Contact stiffness**:The American Association of State Highway Transportation Officials (AASHTO, 1998)
- **Impulse-momentum**: The Federal Emergency Management Agency (FEMA, 1995)[11], U.S. Army Corps of Engineers, 1995)[10]
- **Work-energy**: The National Association of Australian State Road Authorities(NAASRA, 1990).

The aim of each approach is to estimate the maximum vessel impact force based on the velocity and total mass of vessel. Each requires an additional parameter: impulse-momentum requires the stopping time; work-energy, the stopping distance; and contact stiffness, the effective contact stiffness. In the following section we develop a one-degree-of-freedom model of the impact between vessel and a bridge structure. We review each of the above approaches and discuss the assumptions required by each for estimating the maximum vessel impact force. We then discuss other influences that can affect the vessel impact force: added mass and the vessel orientation on impact.

2.1 One-Degree-of-Freedom Model

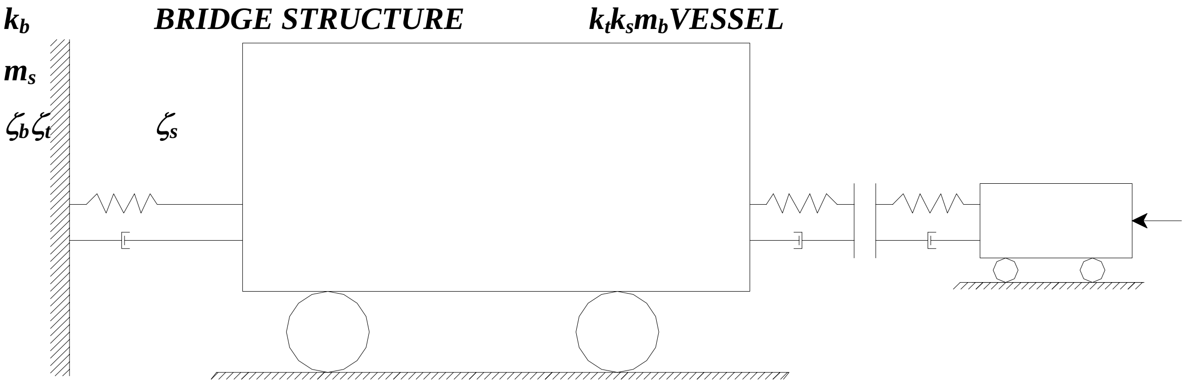

Fig.2. Impact of Vessel with the Bridge Structure

a. A vessel impacting a structure can be modeled as shown in Fig.2 where:

m_s is total mass of vessel,
K_s is stiffness (a linear constant of proportionality between penetration depth and force),
ζ is damping coefficient, and
u_s is velocity of vessel.

b. The bridge structure can be modeled as shown in Fig. 2 Where:

K_b is the stiffness of the bridge structure, (associated with the supporting foundation)
m_b is the mass of the bridge structure, and
k_t is The local deformation of the bridge structure at the impact zone.

The vessel has a total mass of m_s and an approach velocity of us. The elastic deformation of the vessel at impact is described by ks. It is reasonable to assume that *(Robert and steven, 2002)*[7]:

- The collision occurs over such a short duration that damping can be neglected.
- This system can be reduced to one degree of freedom if the bridge structure can be considered to be rigid. If $k_b >> k_s$, that is, the bridge structure support stiffness is much greater than the stiffness of the target zone or the vessel.
- The bridge structure will also act as if it is rigid if the mass of the bridge structure is so great that it doesn't move appreciably in response to the impact of the vessel.

The descriptive equation of such a one-degree-of-freedom model is

$$m_s \ddot{x} + \hat{k} x = 0 \qquad \text{Eq.1}$$

Where k is the effective contact stiffness of the collision

$$\hat{k}^{-1} = \frac{1}{k_s} + \frac{1}{k_b} \qquad \text{Eq.2}$$

The variable x is the summation of the compression of the target face and the vessel during impact and rebound (e.g., x = x_s+ x_t), and the dot notation indicates the time derivative of x. At the moment of contact between the vessel and the bridge structure (i.e., t = 0), x = 0 and x = u_s, so the solution of equation (3.7) is:

$$x = u\sqrt{\frac{m_s}{\hat{k}}}\sin\left(t\sqrt{\frac{\hat{k}}{m_s}}\right) \qquad \text{Eq.3}$$

Given the linear relationship between the penetration depth and the normal force, $F=kx$, the maximum vessel impact force, $F_{i,max}$, predicted using Eq.3 is:

$$F_{i,\max} = u_s \sqrt{\hat{k} m_s} \qquad \text{Eq.4}$$

Thus, the maximum vessel impact force is a function of the impact velocity multiplied by the square root of the product of the effective contact stiffness and the total mass of vessel.

Note that the maximum vessel impact force is independent of the properties of the bridge structure if the bridge structure is considered to be rigid.

The analysis of a one-degree-of-freedom system given above is valid for vessel impact provided that both the inertia and the stiffness of the bridge structure are large enough that the bridge structure itself does not move in any appreciable amount in response to the impact. This assumption results in the maximum vessel impact forces for the relatively short impact durations we are investigating. However, a light bridge structure that has soft foundation stiffness will move in response to the impact, and the system can only be accurately described by the two-degree-of-freedom equations of motion; in this case we expect the equations based on the one-degree-of-freedom analysis to over-predict the actual vessel impact force.

2.1 Main approaches for estimating Vessel Impact Force

2.1.1 Contact-Stiffness Approach

Eq.5 has the same form as the expression adopted for calculating vessel impact forces on bridge piers *(AASHTO 1998)*, where the maximum collision force on the pier, F_v, is based on the dead-weight tonnage of the vessel, *DWT* (long tons), and the vessel velocity, *u* (ft/s). The adopted expression for the maximum impact force of a vessel collision, computed in English units and using an empirical coefficient for the stiffness, is:

$$F_{v(kips)} = 8.15u\sqrt{DWT} \qquad \text{Eq.5}$$

We refer to this approach for estimating the maximum impact force as the contact-stiffness approach because it requires only the effective contact stiffness of the collision to estimate the maximum impact force of an impactor with known mass and velocity *(Robert and steven, 2002)*[7].

2.1.2 Impulse-Momentum Approach

The impulse-momentum approach equates the impulse acting on the vessel in contact with the bridge structure with the change in momentum of the vessel. The governing equation for this approach is based on the definition of impulse, I:

$$I = \int F(t)dt = t_i \overline{F}_i = \int d(u_s m_s) \qquad \text{Eq.6}$$

where F is the force acting on the vessel and is a function of time, t, $\overline{F}_i$ is the time-averaged force, and I is equal to the total change in the momentum of the vessel over the course of the impact. Integration of E.6 requires the functional relationship between impact force and time. If we use F_i and assume that the momentum of the vessel goes to zero as a result of the impact, then Eq.6 becomes.

$$\overline{F}_i = \frac{u_s m_s}{t_i} = \frac{u_s w_s}{g t_i} \qquad \text{Eq.7}$$

Where w is the weight of the vessel, and g is the gravitational constant. The impact duration, t_i, is equal to the time between the initial contact of the vessel with the bridge structure and the maximum impact force. An independent estimate of t_i is required to estimate the impact force. The impulse-momentum approach has been adopted by FEMA (1995)[11] and the U.S. Army Corps of Engineers (1995)[10].*[For brevity we refer to these works collectively as FEMA guidance since the FEMA publication, Engineering Principles and Practices for Retrofitting Flood-prone Residential Buildings (FEMA, 1995)*[11]*, provides the most comprehensive description of applying the design approach.]* Eq.7 is the expression used in the FEMA guidance. FEMA suggests that a value of 1 sec be used for t_i.

A limitation of Eq.7 is that it gives the average impact force, not the maximum force, an important point that is not explicitly stated in the FEMA (1995)[11] guidance. An expression for the maximum force, $F_{i,max}$, can be obtained if the function of the force with time, F(t), is assumed. A linear rise of force with time would be the simplest approach. However, based on Eq.3 we would expect that the functional dependence of force on time is sinusoidal, which results in *(Robert and steven, 2002)*[7]:

$$F_{i,\max} = \frac{\pi u_s m_s}{2 t_i} \qquad \text{Eq.8}$$

2.1.3 Work-Energy Approach

In this case the impact force is computed by equating the work done on the bridge structure with the kinetic energy of the vessel and assuming that the velocity of the vessel goes to zero as a result of the collision *(Robert and steven, 2002)* [7]:

$$W = \int F(x)dx = \int d\left(\frac{1}{2}mu^2\right) \qquad \text{Eq.9}$$

where W is the work done by the change in kinetic energy, $½mu^2$. The force is a function of the distance, x, over which it acts *(F = kx)*. We define S, the stopping distance of the vessel, as the distance the vessel travels from the point of contact with the target until the vessel is fully stopped (u = 0). Then Eq.9 can be solved as follows:

$$\int_0^s kxdx = \frac{1}{2}mu_0^2 \qquad \text{Eq.10}$$

Or

$$kS^2 = mu_0^2 \qquad \text{Eq.11}$$

Since $F_{i,max} = kS$, equation (3.17) becomes:

$$F_{i,\max} = \frac{mu_0^2}{S} = \frac{wu_0^2}{gS} \text{ or } F_{i,\max} = \frac{2}{S}KE \qquad \text{Eq.12}$$

This is the expression used by NAASRA (1990) to compute impact forces of vessel or woody debris on bridge piers.

Fenske,.1995) proposed a formulation nearly identical to Eq.12 except that a coefficient, C_f, is introduced to account for variations in the "stiffness of the bridge, relative angle of impact, fluid damping and [pier] mass:

$$F_{i,\max} = C_f \frac{mu^2}{S} \qquad \text{Eq.13}$$

However, appropriate values of C_f and S were not presented in that work.

2.1.4 Equivalence of Approaches

Though the above analyses of the maximum impact force are presented as three separate approaches, the one-degree-of-freedom model can be used to demonstrate that they are ,equivalent. We can use Eq.3 to determine the values of t_i and S that coincide with $F_{i,max}$. These are the values required by Eqs.8 and 12 *(Robert and steve,2002).*

$$t_i = \frac{\pi}{2}\sqrt{\frac{m_s}{\hat{k}}} \qquad \text{Eq.14}$$

And

$$S = u_s\sqrt{\frac{m_s}{\hat{k}}} \qquad \text{Eq.15}$$

Substituting Eq.14 into Eq.8 or eq.15 into Eq.12 yields

$$F_{i,\max} = u_s\sqrt{\hat{k}\,m_s} \qquad \text{Eq.16}$$

Which is identical to Eq. 4.

Eqs.14 and 15 show that impact duration and stopping distance are not constants that are independent of the properties of the vessel involved in the collisions. Indeed, the impact duration depends on the total mass of the vessel and the contact stiffness of the interaction, while the stopping distance depends on the approach velocity as well as the vessel mass and the contact stiffness. Treatment of ti and S as constants that are independent of debris mass and velocity has led to the disparate estimates of impact forces using these otherwise equivalent expressions.

In this study, the estimation of vessel impact force was according to the contact-stiffness approach.

2.2 Factors acting on the value of Vessel Impact Force

The analyses described above implicitly assume that the mass of the vessel is uniformly affected by the collision, and it does not account for added mass or vessel orientation. The mass of the vessel will not be uniformly affected in collisions that cause the vessel to rotate or merely redirect the trajectory of the vessel. Eccentric and oblique collisions tend to cause rotation of the vessel. Collisions perpendicular to the long axis of "long" vessel can cause the ends of the vessel to rotate as a result of flexure. We may expect the maximum impact force to be increased by added mass and decreased through oblique and eccentric collisions.

$$\frac{F_{i,\max}}{F_{i,\max}^{90}} = \frac{1}{\sqrt{1+\left(\frac{\varepsilon_0}{r_i}\right)\left(1+\mu\frac{r_0}{\varepsilon_0}\right)}} \qquad \text{Eq.18}$$

2.3 AASHTO Guide Specification Design Method (II)

The AASHTO Guide Specification (1991)[1] defines the acceptance criteria for two bridge classifications: regular and critical bridges. For regular bridges, the acceptable annual frequency of collapse for the total bridge elements, AF_c, should be equal to, or less than, 0.1 in 100 years. For critical bridges, the acceptable annual frequency of collapse, AFc, should be equal to, or less than, 0.01 in 100 years.

According to the design Method II, the annual collapse frequency of the j th bridge component shall be computed by

$$AF_{c_j} = \sum_{i=1}^{n} N_i . PA_i . PG_{i,j} . PC_{i,j} \qquad \text{Eq.19}$$

Where:

AF_{cj} is annual frequency of the*jth*bridge component collapse due to vessel collision, *j = 1, …, m,* and

m is the total number of bridge components susceptible to vessel collision;

N_i is annual number of the*ith*vessel category classified by type, size, and loading condition which can strike the bridge element, *i = 1, …, n,* and

n is the total number of classified vessel categories;

PA_i is probability of vessel aberrancy of the*ith*vessel category;

PGi,j is geometric probability of a collision by an aberrant vessel in the.Wang and Liu 2 ith category with the jth bridge component; and

PCi,j is probability of the *jth* bridge component collapse due to a collision with an aberrant vessel in the *ith* category.

2.1 The Parameters Required for Estimating the Vessel Impact Force

The vessel impact force in direct head-on collision with a bridge pier can be approximated by the formula: $F_{v(kips)} = 8.15u\sqrt{DWT}$

Where: *u* is vessel speed (ft/sec),
DWT is deadweight tonnage of vessel (tons).

This formula requires some parameters, where:

- Vessel speed near bridge (3.08 m/sec up to maximum speed 18 m/sec. *(Bangash, 1993).*
- Loading conditions (fully-load vessel and unloaded vessel), (2000 ton up to 96386 ton).
- Vessel impact force was to be assumed to be a static load applied at the water level *(Vijay Chandra and Szecsei, 1980).*
- Draft and trim, (for dynamic analysis only and included in constant factor for static analysis).

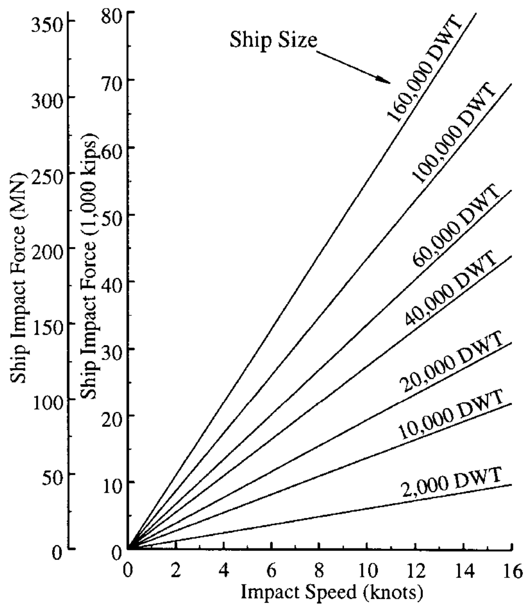

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Fig.3. typical vessel impact forces

Finally, for bridges crossing over navigable waters, impact of vessels shall be taken into account in the design of piers and pier foundations, or adequate remote protection devices shall be provided. The number of piers to be designed against impact or requiring protection as well as the size of vessels to be considered in the calculation of collision forces shall be determined by the Concessionaire in consultation with the navigational authorities. Piers and pier foundations shall be designed to resist direct impact, head-on, oblique and sideways impacts by fully-laden vessels and unloaded vessels.

The bridge superstructure shall be designed to resist a local ultimate horizontal design force of 50 kN representing a collision between a vessel's antennae structure and the bridge superstructure unless a survey of vessels effectuated by the successful Proponent or the local Authorities indicate otherwise *(AASHTO, 1998).*

3. Dynamic Analysis

Many dynamic structural analysis problems require the engineer/analyst to prescribe time-varying parameters such as load, displacement or time histories of ground acceleration. However, in some cases such parameters cannot be determined ahead of time. For dynamic Bridge structure analysis under vessel impact, the impact load is a function of the structure and soil characteristics and is therefore unknown prior to analysis *(Consolazio and Cowan, 2005).*

4. Case study

There are many cable-stayed bridges that could be chosen as a case studies, in order to investigate the previously mentioned iterative technique. However, it may be more convenient to choose a general and realistic case. For this reason, the case study presented here is as close as possible to the Tatara Cable-Stayed Bridge:

The "Tatara Bridge" is cable stayed bridge, whose 890 m center span is longer than that of the "Normandy Bridge" in France by 34 m. (Fig. 4, 5)show the general arrangement of the Tatara Bridge"‹while the main tower and the main girder section are shown in (Fig. 6,7), respectively. *(Honshu-Shikoku, 1996)*

The main tower is 220 m high and designed as an inverted Y shape. It has a cross-shaped section with corners cut for higher wind stability and better landscaping. (Material properties, G= 8.10E+06 t/m2, E= 2.10E+07 t/m2, TC= 1.20E-05).

The main girder section consists of three spans, 270 m, 890 m, and 320 m, and measures 1480 m in total length. As either side span is shorter than the center span, PC girders are installed at each end of both side span sections as counterweight girders to resist negative reaction. This cable stayed bridge thus uses a steel and PC connection girder . The bridge has a total width of 30.6 m, including a road for motorized bicycles and pedestrians (hereafter called sidewalk) and a girder height of 2.7 m. It uses flat box girders attached with fairings to ensure wind stability. (prestressed concrete sections properties,G= 1.22E+06 t/m2, E= 2.80E+06 t/m2, TC= 1.00E-05 and steel sections properties, G= 8.10E+06 t/m2, E= 2.10E+07 t/m2, TC = 1.20E-05).

Cables installed in 21 levels were two-plane multi-fan cables (maximum cable length: about 460 m.(Cabl e s of the br idge have indent ed sur f a c e s in the polyethylene cable coating, similar to dimples on a golf ball, to resist vibration caused by both windy and rainy weather (rain vibration). (Material Properties of the Cables, E= 2.00E+07 t/m2, TC= 1.20E-05).

Different codes were adopted to cover all aspects. At first, the overall stability of the girder, considering different modes of instability was checked for each section of the girder by utilizing the results of an eignvalue analysis. Then, the ultimate capacity of the whole section was checked by, adopting an interaction equation of the Japanese code (JSCE). 1987 (Attia, 1997).

The ultimate strength of the flange has been evaluated based on British code (5400), 1983. Meanwhile, the ultimate strength of the web has been checked by equations of the American code (AISC), 1978. Furthermore, a large deformation analysis was performed to compare its results with the results of elastic analysis.

The complete three-dimensional finite element model for Tatara cable-stayed bridge was developed to similar to the Japanese model. Shells and Frames element were used to model the bridge elements in a fish-bone style (Fig.8).

Fig. 4. Tatara Cable-Stayed Bridge

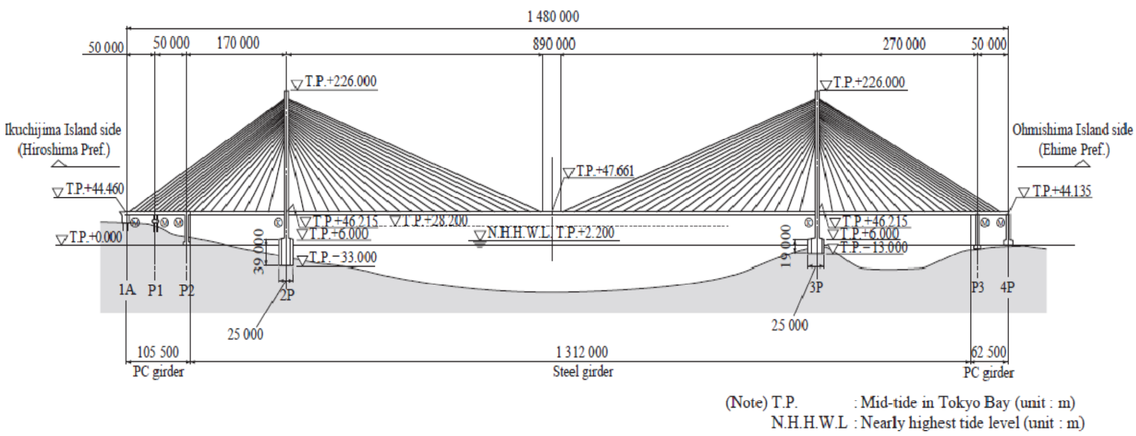

Fig. 5. General arrangement Tatara Cable-Stayed Bridge

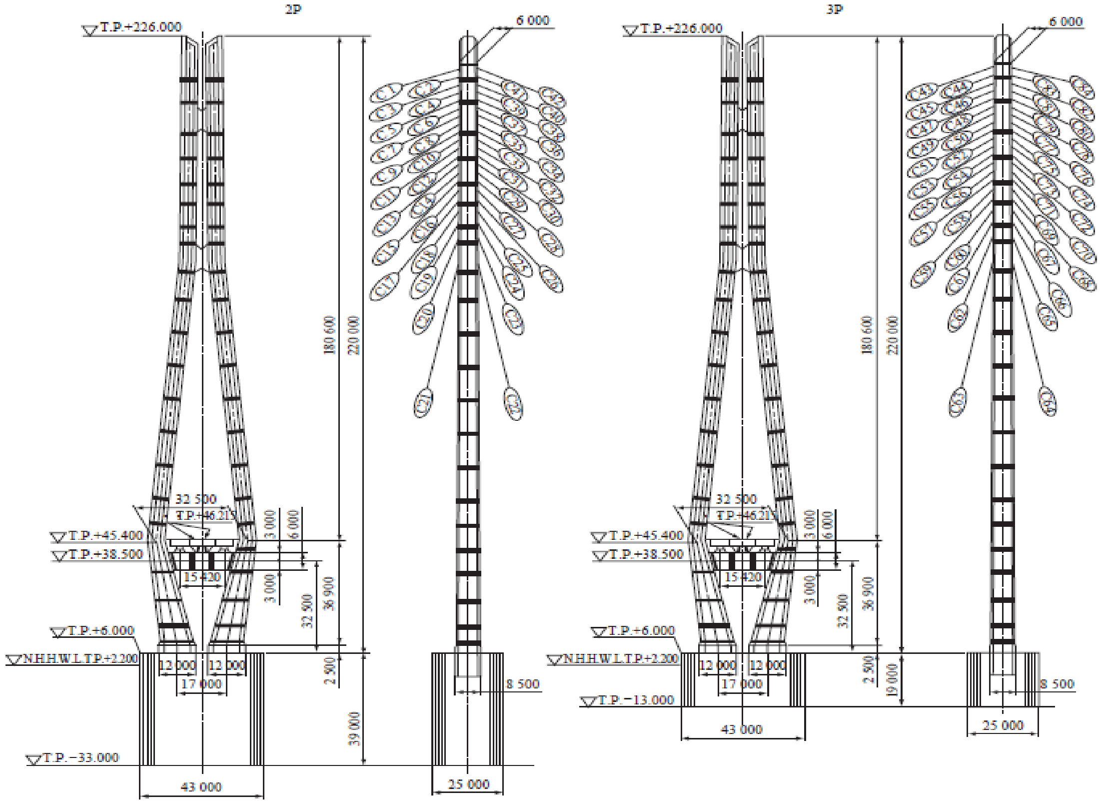

Fig.6. General arrangement (Main Tower)

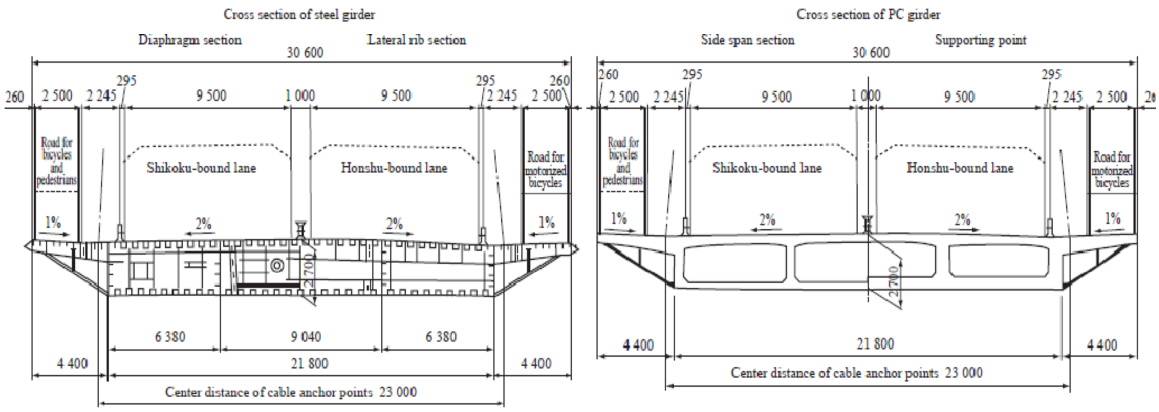

Fig.7. General arrangement (Main Girder)

Fig.8. Three-Dimensional Finite Element Model of Tatara Bridge

5. Results and Discussion

A comparative study has been conducted between the results of the gravitational dead loads and the initial prestressing cable force for each model and the different cases of loading for the impact loads. The effects of the vessel velocity, deadweight tonnage on the displacements of the bridge girder and the tower was investigated for each studied bridge.

The gravitational dead loads and the initial prestressing cable force has the basic case of loading of the Tatara Cable-Stayed Bridge according to analyzing a three-dimensional model and the values of displacements of studied cases of loading for the impact loads are shown in the following figures

For evaluation of the effects of the vessel velocity and deadweight tonnage of the vessel on the displacement of the bridge girder and the tower with respect to the increase of the bridge span is considered in this thesis

The results of analysis showed that the effect of the factors in the impact load formula on the displacements of the main girder of the cable-stayed bridges increases with the increasing in the velocity and dead weight tonnage. In addition, the effect of the factors in the impact load formula on the longitudinal displacements of the top tower of the cable-stayed bridges increases with the increasing in the velocity and dead weight tonnage, but the effect of the factors in the impact load formula on the vertical but transversal displacements of the top tower of the cable-stayed bridges independent with the increasing in the velocity and dead weight tonnage.

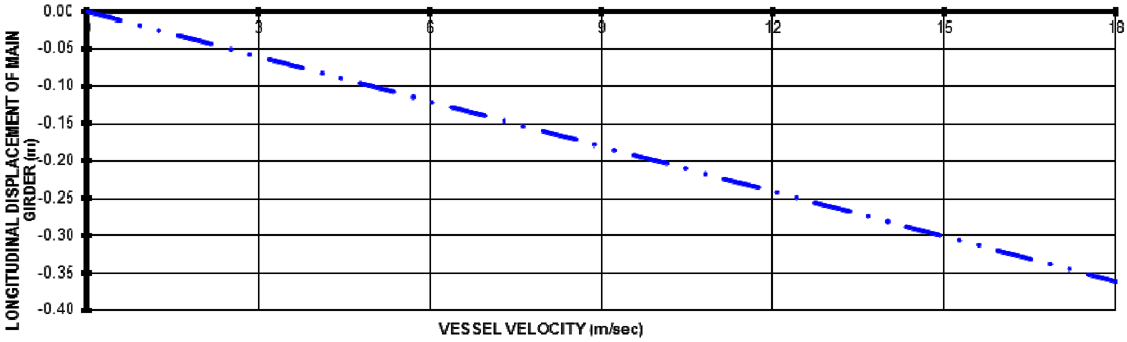

Fig.9. Effect of the Vessel Velocity on the Longitudinal Displacements at Midpoint of the Main Girder of the studied models

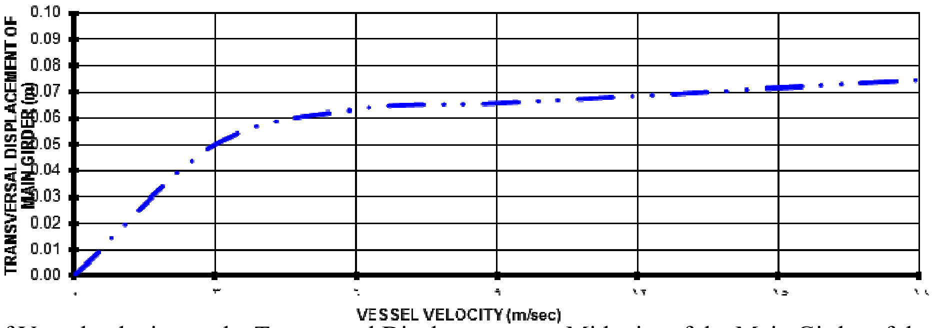

Fig.10. Effect of Vessel velocity on the Transversal Displacements at Midpoint of the Main Girder of the studied models

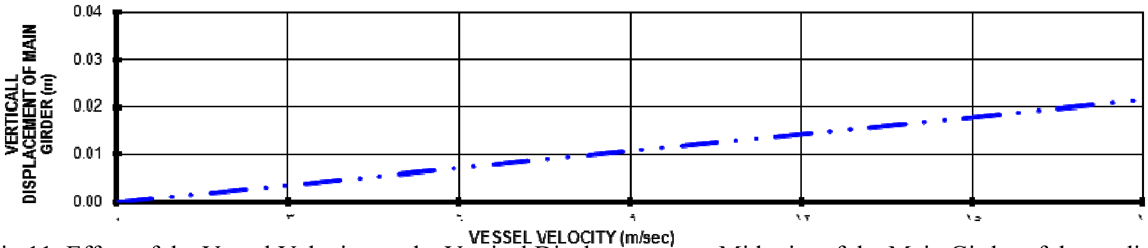

Fig.11. Effect of the Vessel Velocity on the Vertical Displacements at Midpoint of the Main Girder of the studied models

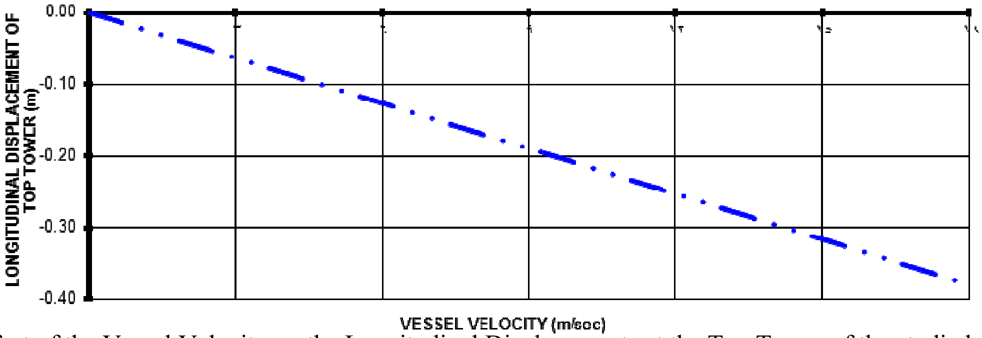

Fig.12. Effect of the Vessel Velocity on the Longitudinal Displacements at the Top Tower of the studied models

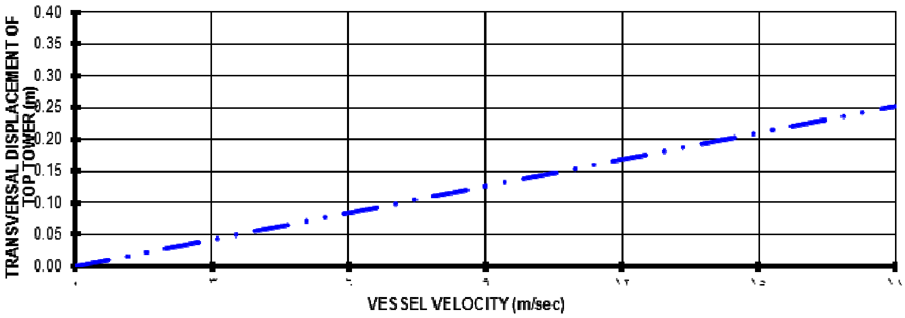

Fig.13. Effect of the Vessel Velocity on the Transversal Displacements at the Top Tower of the studied models

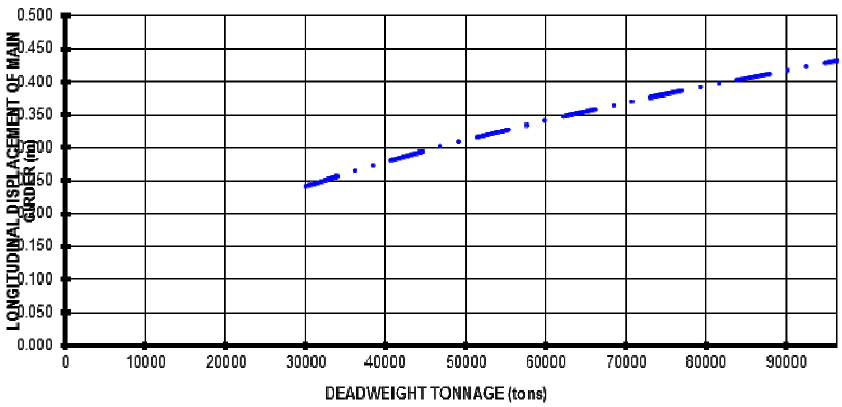

Fig.14. Effect of the Deadweight Tonnage on the Longitudinal Displacements at Midpoint of the Main Girder of the studied models

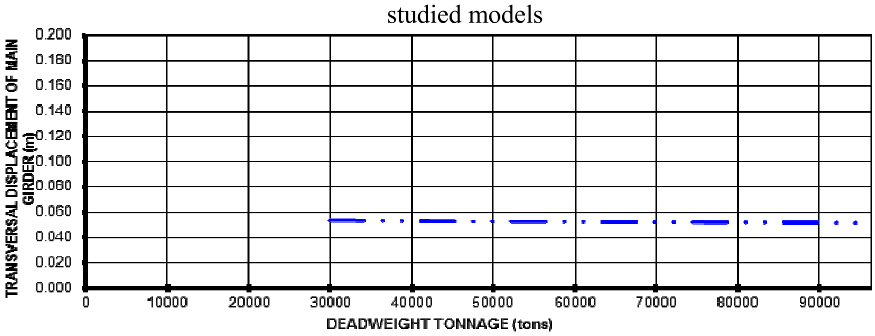

Fig.15. Effect of the Deadweight Tonnage on the Transversal Displacements at Midpoint of the Main Girder of the studied models

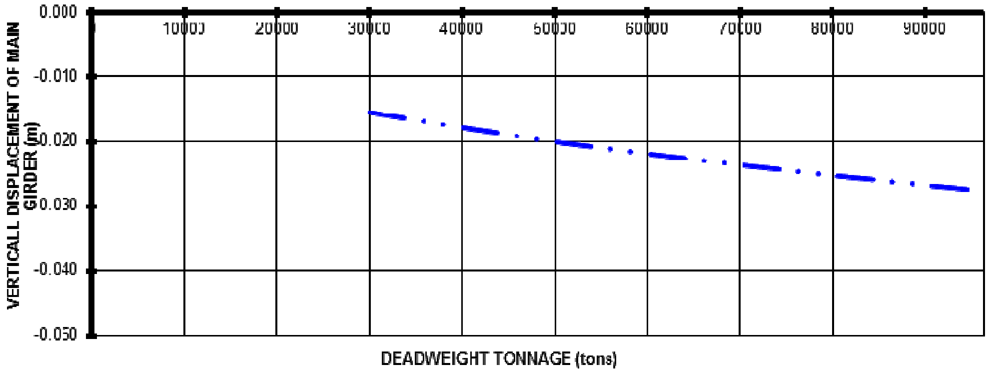

Fig.16. Effect of the Deadweight Tonnage on the Vertical Displacements at Midpoint of the Main Girder of the studied models

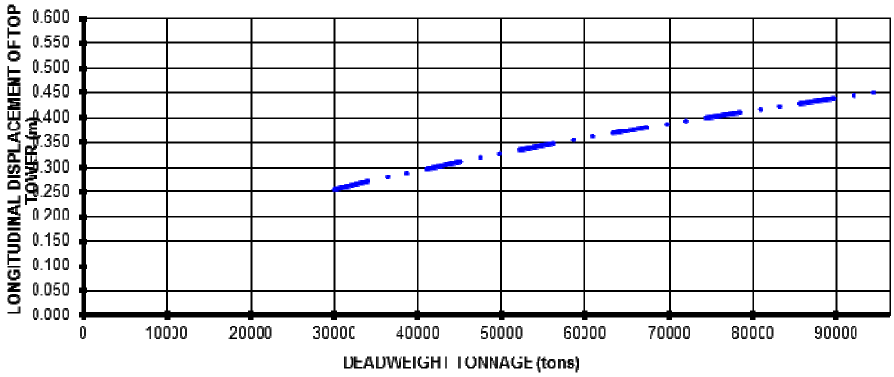

Fig.17. Effect of the Deadweight Tonnage on the Longitudinal Displacements at the Top Tower of the studied models

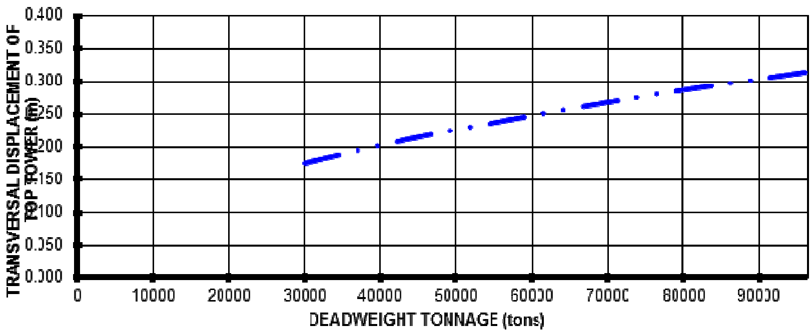

Fig.18. Effect of the Deadweight Tonnage on the Transversal Displacements at the Top Tower of the studied models

6. Conclusions

The current world trend is to adopt long-span cable-supported bridges for wide water crossing. Over the past 40 years, rapid developments have been made on long-span cable-supported bridges. Cable-stayed bridges are now entering a new era, reaching central span lengths up to 1,000 m and even longer. On the other hand, suspension bridges central span lengths reached up to 2,000 m and even longer. This research is concerned with the effect of vessel impact forces on Tatara cable-stayed bridge. A comparative study was conducted to investigate the effect of vessel impact force on the bridge deformations partical for the central span and top of the tower of the bridge structure. The Contact-Stiffness Approach - AASHTO, 1998 – Formula was applied to determine the maximum impact force of a vessel collision as a function of the vessel velocity and the deadweight tonnage of the vessel. Impact force is applied to the tower of bridge at the point above water level.

Excluding the original basic case of loading model of studied bridge, cases of loading with different values of the vessel velocity, and deadweight tonnage of the vessel were studied for bridge.The three-dimensional finite element models of Tatara cable-stayed bridge under action of dead loads, which contribute the most to total bridge loads were analyzed. The finite element models were developed and generalized in a fish bone style mesh diagram. Shells and Frame elements were used to model the bridge elements. The geometrical nonlinear behavior due to cable sag and soil-structure in terms of were neglected. Piers were modeled as hinged supports. Towers were considered fixed to foundation. The three displacements of the midpoint of main girder and the top tower corresponding to different cases of loading were obtained and the results were compared for Tatara cable-stayed bridge. Based on the comparative study , the following conclusion can be drawn as follows:

1- Vessel velocity can influence linear on the value of the ship impact force where it has a maximum effect when it is the maximum value 18 m/sec.
2- Dead weight tonnage can influence linear on the value of the ship impact where it has a maximum effect when it is the maximum value.
3- Collision angle can influence nonlinear on the value of the ship impact force where it has a maximum effect when the ship is head on with the bridge axis (at $\alpha = 0$ o).
4- A linear relations is remarked between the vessel velocity and the longitudinal displacement of the main girder is increasing linear, and may be neglected in other direction.
5- A linear relations is remarked between the vessel velocity, the longitudinal and transversal displacements of the top tower is increasing linear, and may be neglected in the vertical displacement.
6- A linear relation is remarked between the deadweight tonnage of the vessel and the longitudinal displacement of the main girder is increasing linear, and may be neglected in other direction.
7- A linear relation is remarked between the deadweight tonnage of the vessel, the longitudinal and transversal displacements of the top tower is increasing linear, and may be neglected in the vertical displacement.

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