Comparison of pier bridge design between AASHTO and Chinese codes

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Abstract. Many codes are used in different countries to design different constructions including bridges. Using of suitable code is depending on the project requirements and safety. In this paper two different codes are used to design a pier bridge to show the differences and similarities between them in order to use the suitable one according to the needs of this design. The AASHTO code shows more safety compared to the Chinese code except pier design in bending and shear due the conditions in US country which needs more restricted conditions compared with that used in China. This has a considerable effect in Iraq to find the best code to be used because there is no an available special code. Another effect is taken into consideration in this study which is the effect of bridge pier collision due to impact loads which is necessary to show the suitability of the bridge design using the mentioned codes. This effect shows the need of more requirements in the design to be safe and resist the lateral loads effect.

Keywords: AASHTO code; bridge engineering; Chinese code.

1. Introduction

Column pier design takes into account all the vertical and horizontal loads imposed by the superstructure including the influence of earthquakes [1]. There are other factors must take into account in the design, which are of the serious effects caused the failure of bridge. These effects are the collision of vehicles or vessels with the pier as a result of many reasons and therefore this collision will lead to a sudden failure of pier. Therefore, in order to protect the pier from these effects, it should be provided adequate protection or including the requirements of design for this aspect. Owing to the high strength and ductility and low permeability of reactive powder concrete, so it used for purpose of protection [2, 3] and which result in substantial reinforcement savings, thus advancing the objectives of sustainability in construction [4]. There are other factors and other influences such as the influence of ice and other harsh weather conditions, etc. [5].
Researchers investigated and reported on 19 accidents involving trucks colliding with bridge piers [6]. A forward-looking infrared (FLIR) video surveillance system is presented for collision avoidance of moving ships to bridge piers [7].

Bridge structures must not only be designed to resist gravity, wind, and earthquake loads, but must also be capable to resist ship and lateral loads [8]. Nowadays the development of city traffic is rapid. The vessel collision force protection based on the probability method in AASHTO LRFD. Vessel-bridge collisions occur many times, which cause problems to the bridge’s operation and ships [9].

The forces of ice became the primary side load effect on the sidewalks, and the subject of uncertainty and controversy. The final preparation of the piers is designed to reduce snow forces and thus meet the reliability required for lateral stability and to minimize potential impacts on the ice system. Several years of observations and measurements provided ample evidence of design effectiveness on both sides, and the design may be more active than initially considered. A whole monitoring program has been done to assess the bridge’s performance in a number of areas, including ice forces and ice interactions. The pavement arrangement was effective in reducing ice forces, the nature of ice interactions with pavements, and the resulted forces are such interactions [10].

2. Models for study

The pier model is used in the design. The dimensions of pier were obtained from structural plans of a bridge which its foundation had been previously designed in another study [11, 12]. The pier, hereafter referred in Figure 1, is a 2m in diameter, and width is 4m. The diameter of drop panel of pier cap is 4m for both sides with top cap thickness of 20cm and lateral width 6.28m. The total height of pier with the drop panel is 7.2m. Bearing pad dimensions were (1.4×2.5) m with thickness 37cm. The pier is attached to a reinforced concrete pile hat dimensions are (5×6) m and thickness 1.5m that is embeded 1.5 m under-ground. The superstructure, cap, and pier are supported by four 1 m dia. concrete bored piles 62.7 m in length. The properties of box girder are taken as the standard beam shown in Figure according to AASHTO code [13, 14] with 32m length, the weight is 7862 kN. Super-imposed dead load is 3791 kN. So, the total applied dead load of the structure on the pier is 11718 kN including the loads of bearings. Pier will be designed by the use of two codes. The first code is the AASHTO code and the second is the Chinese code [15] in order to note the difference in design and knowledge of the code that is best for design and provides greater protection as a result of the effect of external influences such as the collision of vehicles in pier. The vehicle model is a 66 kN Ford truck and the ship model [16].

3. The design information

The design according to Chinese code is taking span girders bridge, beam length of 32.6m, the bridge width 12m.

Dead load: weight of beam 7862.88kN, two horizontal loads for the 3.792MN
Materials: Concrete used C30, the main strength index is shown in Fig. 2.

\[ f_c = 20 \text{MPa}, \quad f_{ct} = 2.2 \text{MPa} \]

Modulus \[ E_c = 3.2 \times 10^4 \text{MPa} \]

Section: \( d=2\text{m}, \quad \text{Lateral width} = 4\text{m}, \quad A_k = 71415.926 \text{cm}^2 \)

The area of longitudinal reinforcement elements is not less than 0.5%, not exceed than 3% of cross-sectional area. Calculated at 1%, \( A_g = 714.16 \text{cm}^2 \). Selected 116Φ28HRB400. The total area of longitudinal reinforcement 714.27cm²; Static distance of longitudinal reinforcement \((\pi \times 200+400)/116 = 8.865\text{cm}\)

Greater than 5cm, less than 30cm; cover layer thickness: 5cm. Stirrups: optional Φ8, Spacing 35cm, less than 15 times the diameter of longitudinal reinforcement.
3.1. Dead load calculation

Beam weight 7862.88kN, two horizontal loads for the 3792KN, dead load pressure:

\[ N_1 = 7862.88 + 3792 = 11654.88 \text{kN} \]

Pier cap unfactored dead load

\[ W_{\text{bearings}} = 1.4 \times 2.5 \times 0.37 \times 24 \times 2 = 62.16 \text{kN} \]

\[ w_{\text{cap}} = (\text{unit weight of concrete}) \times (\text{cap cross-sectional area}) \]

\[ = \frac{6.28 + 4}{2} \times 1 \times 4 \times 24 = 493.44 \text{kN} \]

Unfactored column dead load

\[ W_{\text{column}} = (\text{unit weight of concrete}) \times (\text{column cross sectional area}) \]

\[ = 2 \times 2 \times 6 \times 24 + \frac{\pi}{4} \times 2^2 \times 6 \times 24 \]

\[ = 1028.38 \text{kN} \]

Unfactored footing dead load

\[ W_{\text{footing}} = (\text{unit weight of concrete}) \times (\text{footing cross sectional area}) \]

\[ = 5 \times 6 \times 1.5 \times 24 \]

\[ = 1080 \text{kN} \]
Total dead load = 11654.88 + 62.16 + 493.44 + 1028.38 + 1080 = 14318.86kN

3.2 Vertical static live load

Live Load Diagram the most unfavorable light load for the single-hole, heavy hole, double holes empty overloaded and live load, respectively, are calculated as follows:

(1) hole at light loads, such as the right Figure (2A) shows

According to the knowledge of structural mechanics, pivot counter-force:

\[ R_1 = \frac{1}{32} [220 \times 5 \times (3 - 0.3) + 25.1 \times 92 \times (25.1/2 + 32 - 0.3)] = 3286.0 \text{ kN} \]

Torque on the center pier is:

\[ M_{R1} = 3286 \times (0.3 + 0.1) = 1314.4 \text{ kN.m} \]

(2), heavy hole, live load arrangement shown in Figure (2B) shows Fulcrum reaction forces

\[ R_2 = \frac{1}{32} [25.1 \times 92 \times (25.1/2 - 0.3) + 220 \times 5 \times (32.6 - 3 - 0.3)] = 1891.2 \text{ kN} \]

The Torque on the center pier

\[ M_{R2} = R_2 \times 0.3 = 1891.2 \times 0.3 = 567.36 \text{ kN.m} \]

heavy holes, live load arrangement is shown in Figure (2C)

According to the knowledge of structural mechanics, G1/L1 = G2/L2, the fulcrum against the greatest force, calculate X

\[ (7.5 + X - 2.6) \times 92 + (32.6 - 7.5 - X) \times 92 = (32.6 - 7.5 - X) \times 92 + 220 \times 5 \]

\[ X = 7.14 \text{ m}, \ G1 = G2 = 2752.3 \]

\[ R_3 = \frac{1}{32} [20.56 \times 80 \times 20.56 / 2 + 12.04 \times 92 (12.04 / 2 + 20.56)] = 1448.5 \text{ kN} \]

\[ R_4 = \frac{1}{32} [220 \times 5 \times 10.14 + 17.96 \times 92 (7.5 + 7.14 + 17.96 / 2)] = 1568.2 \text{ kN} \]

\[ R_{3-4} = R_3 + R_4 = 3016.7 \text{ kN} \]

Moment of the center of the pier:

\[ M_{R_{3-4}} = (R_3 - R_4) \times 0.4 = 47.88 \text{ kN.m} \]

(4) Empty holes live load, live load arrangement shown in Figure (2D)

\[ R = 2 R_5 = 2 \times (10 \times 32.7 / 2) = 327 \text{ kN} \]

Moment of the center of the pier:

\[ M_R = 0 \]
3.3 Braking force

A hole light-load and overload of the beam the same vertical static live load, the braking force in accordance with the vertical live load of 0.1 times the static calculation:

\[ P_t = (5 \times 220 + 92 \times 25.1) \times 0.1 = 340.92 \text{ kN} \]

P, Cross-section of the pier at the bottom of the torque is:

\[ M_{pt} = P_t \times H = 340.92 \times 6 = 2045.5 \text{ kN.m} \]

Two holes heavy braking force

Holes left for the fixed support beam power transmission system

\[ P_{t1} = R_3 \times 0.1 \times 100\% = 144.85\text{ kN} \]

The right hole for the slide plate support beam delivery system of power is:

\[ P_{t2} = R_4 \times 0.1 \times 50\% = 78.4\text{ kN} \]

Reached the braking force on the pier are:

\[ P_t = P_{t1} + P_{t2} = 144.85 + 78.4 = 223.25 \text{ kN} < 340.92 \text{ kN} = P_{t_{\text{max}}} \]

Where \( P_{t_{\text{max}}} \) the hole by the fixed support beam to pass at full load braking force, and its value with the single-hole hole light load or heavy braking or when the traction equal. Therefore, \( P_t \) on the pier at the end of the torque.

\[ M_{pt} = 223.25 \times 6 = 1339.5 \text{ kN.m} \]

3.4 Lateral load

The force of the train rocking and seismic forces considered as a 12% of vertical loads.

3.4.1 Checking the stability of the pier cross-section

Pier bottom section properties

Area: \( A_1 = 71415.926 \text{ cm}^2 \)

Moment of inertia: \[ I_y = \frac{\pi}{64} d^4 + bh^3/12 = \frac{\pi}{64} 2^4 + 2 \times \frac{2^3}{12} = 2.12 \text{ m}^4 \]

Resistance moment: \[ W_y = I_y/(0.5d) = 2.12 \text{ m}^4 \]

Figure 2. models of girder
3.4.2 Stability Checking

"Standard" provides concrete and solid stone pier, pier pressure checking the stability of the following formula: \[ KN < N_{cr} \]

K-factor of safety
N-axial pressure at the pier top
\( N_{cr} \)-Critical load compression
\[ N_{cr} = \frac{4mEI_d}{l_0^2} \]

Consider the impact of variable cross-section, eccentric compression stiffness reduction and the reduced elastic modulus
\[ N_{cr} = \alpha \frac{4mEI_d}{l_0^2} \left[ \frac{1}{1 - \alpha} \frac{1}{1.1R_c A_0} \right] \]

Where
\( \alpha \)— Stiffness correction factor, \( \alpha = \frac{0.1}{0.24 + \frac{2}{k}} + 0.16 \)

\( m \)-variable cross-section of coefficients, look-up Table 1 available here for the cross section, \( m = 1 \)

\( l_0 \)-member cross-section around the bottom of the vertical bending direction of the entire spindle-shaped cross-section moment of inertia

\( L_0 \)-length, \( l_0 = \mu l \), Here \( \mu = 2 \)

\( R_c \)— Ultimate compressive strength, \( R_c = K[\sigma] \), \( K \) For the safety factor

The results of the above checking can be shown in Table 1:

| Live load case | Single-hole light-load | Single-hole light-load | Heavy holes |
|----------------|------------------------|------------------------|-------------|
| Force or torque | \( N \) (kN) | \( M \) (KN.m) | \( N \) (kN) | \( M \) (KN.m) | \( N \) (kN) | \( M \) (KN.m) |
| Cross the bridge dead | 14318.86 | 14318.86 | | | 14318.86 |
| Main load N | | | | | |
| Live load stress R | 3286 | 1314.4 | 1891.2 | 576.4 | 3016.7 | 47.88 |
| Pier together | 17604.86 | 1314.4 | 16210.06 | 576.4 | 17335.56 | 47.88 |
| Initial eccentric moment \( c_0 \) (m) | \( \frac{17604.86}{1314.4} = 0.0746 \) | \( \frac{576.4}{16210.06} = 0.0355 \) | \( \frac{47.88}{17335.56} = 0.0027 \) | | |
| Sectional area \( A_0 \) (m²) | | | | | 7.1415 |
| Moment of inertia \( I_0 \) (m⁴) | | | | | 2.12 |
| Length \( l_0 \) (m) | | | | | 2×6×12 |
| \( E_0 \) (MPa) | | | | | \( 3.2\times10^4 \) |
The results show that in the table: pier pressure stability overloaded with two holes plus dead load of the main bridge span combination of control, a large safety margin, the section does not control the pier design.

4. Checking the pier cross-section and eccentric strength checking

Tables 2 and 3 show the results of the following:

4.1 Strength checking formula

Here is one-way eccentric, strength check calculation formula:

\[ \sigma = \frac{N+G}{A} \pm \frac{M_y \eta \cdot x}{I_y} \leq [\sigma] \]

N-acting on the pier top surface of the axial pressure Section
G-Checking the weight above the pier along the axial

\([\sigma]\) -- Center or eccentric compression pressure allowable stress, check specifications available

\(M_y\)-- Checking the center section of the y-axis moment

\(I_y\)-- The total cross-section moment of inertia of cross sections around the central axis y of

\(x\) -- Computation of the x-axis coordinate of stress points

\(\eta\) -- Moment magnification factor, the bottom of the largest pier.

Which, \(\eta_{\max} = \frac{1}{1 - \frac{KN}{N_{cr}}}\)

4.2 Calculation of stress redistribution

As concrete and stone masonry tensile strength is low, the tensile stress occurs when the cross-section, the assumption cannot withstand tensile stress masonry and cracking, cracking some out of work, cross sections will be re-distribution of stress, called stress redistribution phenomenon.

The formula for stress redistribution: \(e \leq 0.5S\)

which, \(\sigma_{\max}\) -- Eccentric after stress redistribution when the maximum compressive stress

N/A- the average compressive stress in the whole cross-section

\(\lambda\) -- Round and round end type one-way eccentric stress redistribution factor specifications are available for inspection.
4.3 Eccentric pier checking

\[ e = \frac{\eta M}{N} \leq \lceil e \rceil \]

Symbols above.
Specification sections together eccentricity \( e \) provides as follows:
The main role \( e \leq 0.5S \)
Main + additional force (round section) \( e \leq 0.5S \)
Main + additional force

| Table 2. Stress and eccentric pier section checking (main + additional longitudinal force) |
|---|---|---|---|---|---|---|---|
| Live load case | Single-hole light-load | Heavy hole | Heavy holes |
| Force or torque | \( P \) (KN) | \( M \) (KN) | \( N \) (KN) | \( P \) (KN) | \( M \) (KN) | \( N \) (KN) |
| Main Pier together | 17604.8 | 131 | 16210. | 576.4 | 17335.56 | 47.88 |
| N2 pier weight | 1521.82 | 4.4 | 1521.8 | 2 | |
| Additional Braking force | 340.92 | 204 | 340.92 | 2045. | 223. | 1339.5 |
| Attached to the main + | 19126.6 | 335 | 17731. | 340.92 | 2621. | 18857.38 |
| \( \eta_{\text{max}} M \) | 8 | 9.9 | 88 | 9 | 25 | 8 |
| \( \eta_{\text{max}} M \) | 344 | 7.25 | 417 | 1419.1 |
| Pier bottom sections together | 19126.6 | 7.25 | 88 | 340.92 | 2681. | 18857.38 |
| Force eccentric \( e = \frac{\eta M}{N} \) (m) | 0.18 | 0.151 | 0.075 |

Allow the eccentric [\( e \leq 0.5S \)]

| Sectional area \( A0 \) (m²) | 7.1415 | 7.1415 | 7.1415 |
| Cross resistance moment \( W \) (m²) | 21.2 | 21.2 | 21.2 |
| \( \sigma_1 = N/A \) (kPa) | 2678.244 | 2482.93 | 2640.534 |
| \( \sigma_2 = \frac{\eta M}{W} \) (kPa) | 1626.064 | 1264.82 | 669.41 |
| \( \sigma_{\text{max}} = \sigma_1 + \sigma_2 \) | 4304.308 | 3747.75 | 3309.944 |
| \( \sigma_{\text{min}} = \sigma_1 - \sigma_2 \) | 1052.18 | 1218.11 | 1971.124 |
| Coefficient of stress redist | \( \mu = \frac{a}{R} \) | 2 | 2 | 2 |
| \( a = 4 - 2R \) | 2m | 0.18(2×1) = 0.09 | 0.0755 | 0.0375 |
permission value of stress for C30, it is 10mpa

For the circular section, s = d/2.
Stress units in the table kPa. the permission value of stress for C30, it is 10mpa

Table 3. Computation of sectional stress and eccentricity (main + additional longitudinal force)

| Live load case | Empty holes live load, additional force main + lateral | Car-free bridge, the main + additional force horizontally |
|----------------|--------------------------------------------------------|----------------------------------------------------------|
| Force or torque | N (kN) | P | M | N | P | M |
| Main | Cross the bridge dead load N1 | 11654.88 | | 11654.88 | | |
| | Weight bridge pier N2 | 1521.82 | | 1521.82 | | |
| | Empty live load | 327 | | 0 | | |
| Additio | Vertical static live load by 12% consider | 1620.444 | 9722.664 | 1620.444 | 9722.664 |
| nal force | | | | | | |
| Total | 13503.7 | 1620.444 | 9722.664 | 13176.7 | 1620.444 | 9722.664 |

P, N in units of kN, M units of kN * m

As can be seen from the table, the initial eccentricity $e_0 = 0$. $\alpha = \frac{0.1}{0.2 + 0.1} + 0.16 = 0.66$

$$N_{cr} = \alpha \frac{4mEI_d}{l_0^2} \left[ \frac{1}{1 - \alpha} \frac{4mEI_d}{l_0^2} \frac{1}{1.1 R_e A_o} \right]$$

$$N_{cr} = 0.66 \times \frac{4 \times 1 \times 3.2 \times 10^7 \times 2.12}{12^2} \left[ \frac{1}{1 + \frac{4 \times 1 \times 3.2 \times 10^7 \times 2.12}{12^2}} \frac{1}{1.1 \times 7.1415 \times 10500} \right]$$

$$= 52156.69kN$$

$$N_{cr} = \alpha \frac{4mEI_d}{l_0^2} = 0.66 \times \frac{4 \times 1 \times 3.2 \times 10^7 \times 2.12}{12^2} = 1243733.33 \text{ kN}$$

$$\eta_y = \frac{1}{1 - \frac{KN}{N_{cr}}} = \frac{1}{1 - \frac{35209.72}{52156.69}} = 1.0291$$
Allow the eccentric horizontal $\varepsilon = \frac{\eta M_y}{N} = \frac{1.0291 \times 9722.664}{13503.7} = 0.740952$

The detailed design of pier and reinforcement can show in the following Figure:

**Figure 3.** Details of reinforcement

5. Results and Discussion

5.1 Results of the Design

Through the application of the manual procedure for the design of the pier, but using different codes as mentioned previously. The effect of earthquakes has been introduced in the design using AASHTO and Chinese code. The results of the design have been included in the Table 4, by which is noted the difference in the design, because the use of two different codes used in a different country. It is noted an increase in reinforcement area needed in order to design the pier with the U.S code compared to that is required the Chinese code. So, the first code takes into consideration the provision requirements of high design, but at the same time, is a non-economic compared to the second code.

This paper will discuss the design and analysis of the above pier model by using the two codes and the numerical analysis by the finite element software. The collision of pier will be taken for the vehicle and
vessels. Table 1 shows the comparison in design between the two codes. The calculation with design using Chinese codes are shown above in this paper, while the procedure of design in AASHTO code is taken according to [17-19].

| Item No. | Comparison Item | AASHTO Code | Chinese Code |
|---------|-----------------|-------------|--------------|
| 1       | Concrete strength | $f'_c = 35$ MPa | $f'_c = 20$ MPa, $f''_c = 2.2$ MPa |
| 2       | Yield strength   | $f_y = 515$ MPa | $f_y = 515$ MPa |
| 3       | Modulus of Elasticity | $E_c = 3.0 \times 10^4$ MPa | $E_c = 3.2 \times 10^4$ MPa |
| 4       | Dead load        | 11654.88 kN | 11654.88 kN |
| 5       | Live load        | 84.158 kN/m | 94.27 kN/m |
| 6       | Pier cap (Bending and flexure reinforcement) | #8@13cm in the short direction | #8@31cm in the short direction |
| 7       | Pier cap design (Shear reinforcement) | #8@35cm | #8@35cm |
| 8       | Pier design (Shear reinforcement) | #9@7.85cm | #9@7.85cm |
| 9       | Pier design (Shear reinforcement) | #8@30.5cm | #8@35cm |

5.2 Results of the Collision Effects
AASHTO-based calculations for barge impact design begin with choosing the design condition (barge type and impact velocity). Factors such as waterway characteristics, types of barge traffic, and bridge importance (critical or normal) go into this selection process. The working conditions are determined, the kinetic energy of the barge is calculated in the form [4]:

\[
KE = C_H W V^2 / 29.2
\]

(1)

KE is the kinetic energy of barge (kip ft), $C_H$ is the coefficient of hydrodynamic mass, $W$ is weight of the vessel (in tons), and $V$ is the speed of impact (ft/s). It is noted that Eq. (1) is simply an empirical version:

\[
KE = 500 C_H M V^2
\]

(2)

Where:

- $KE =$ Vessel impact energy (J), $C_H =$ vessel displacement tonnage M (Mg), hydrodynamic mass modulus $C_H$, vessel impact velocity $V$ (m/s).

Copy, bundle, copy, bundle, bundle, block, copy, copy, pack, copy, block, block, block, block, for ships in empty or lightly loaded condition. The mass of the diameter block and the combined mass. The hydrodynamic mass coefficient, $C_H$, shall be taken as:
- If under keel clearance exceeds 0.5 × draft:
  \[
  C_H = 1.05
  \]

(3)
- If under keel clearance is less than 0.1 × draft:
  \[
  C_H = 1.25
  \]

(4)
CH values were classified from typical. A space should be taken between the bottom of the vessel and the bottom of the stream. In general, it should be based on the maximum velocity that is representative of extreme events, such as floods and other extreme environmental conditions. The witness:

$$KE = 500 \times 1.25 \times 5000 \times 10.5^2 = 344531250 \text{ J} \quad \text{……………………. (5)}$$

In this expression, $$\alpha_B$$ is the depth (ft) of barge crush deformation (depth of penetration of the bridge pier into the bow of the barge), KE is the barge kinetic energy (kip ft) and $$R_B = (B_B/35)$$ where $$B_B$$ is the width of the barge (ft).

Therefore; $$B_B = 35 \text{ ft, } R_B = 1, \alpha_B = 0.305 \text{ ft}$$

$$P_B = \frac{4112\alpha_BR_B}{(1349 + 110\alpha_B)R_B} \quad \alpha_B < 0.34 \text{ ft}$$
$$P_B = 1 \quad \alpha_B \geq 0.34 \text{ ft} \quad \text{…………………….. (7)}$$

Where $$P_B$$ is the static barge equivalent impact load (kips) and $$\alpha_B$$ is the barge crush depth (ft). The crush model represented is illustrated graphically in. Therefore;

$$P_B = 4112 \times 0.305 \times 1 = 1254.2 \text{ kips} = 5700.7 \text{ kN} > \text{ horizontal loads used in the design } = 1398.586 \text{ kN} \quad \text{Not OK, the pier should be design to resist a horizontal load } = 5700.7$$

The pier impact force ship collision shall be taken as:

$$P_s = 1.2 \times 10^5 V(DWT)^{0.5} \quad \text{……………………… (8)}$$

Where:

- $$P_s$$ equ. impact force (N)
- $$DWT$$ tonnage deadweight (Mg)
- $$V$$ impact velocity of vessel (m/sec.)

The recommended equation is:

$$P_{max} = 0.88 \times (DWT)^{0.5/3} (v/8)^{2/3} \quad \text{……………………… (9)}$$

Where: $$P_{max}$$ is the maximum collision force; $$v$$ is the ship speed (m/s)

The maximum force of collision of 5000 tons reaches 89MN of versatile ship, and 100000 tons is 398.45 MN. Where the vessel impact velocity is 10.5 m/sec.

This equation is provided for highway and railway bridges and culverts design.

In most codes, the calculated ship collisions depend on empirical formulas, while the types and bridge piers and sizes of ships are quite different, therefore, the obtained results have quite high difference with the fact ones. The collision forces calculation and the efficacy design and analysis of anti-collision are reasonable.

6. Conclusion

The comparison in design with codes is very necessary for designer to take into consideration the suitable way in design according to the available conditions in the site or location needed to design.

In this study two codes are taken to design a pier bridge and the following conclusions are seen:

1. The AASHTO code in Pier cap (Bending and flexure reinforcement) needs more reinforcement which means it is safer. The Chinese code needs 42% of reinforcement compared with AASHTO code.
2. The AASHTO code in Pier cap design (Shear reinforcement) gives the same reinforcement compared with Chinese code.

3. The AASHTO code in Pier design gives lesser reinforcement compared with Chinese code. Which means that the second is safer in Pier design. AASHTO code needs about 85% of reinforcement compared with Chinese code.

4. The AASHTO code in Pier design (Shear reinforcement) needs more reinforcement which means it safer more than the Chinese code. Chinese code needs about 87% of reinforcement compared with AASHTO code.

5. The study of the effect of bridge pier collision due to impact loads shows this effect takes into consideration in the design requirements.

7. References

[1] Sultan H K Mohammed A T Qasim O A Maula B H Aziz H Y 2020 Ductility Factor Evaluation of Concrete Moment Frame Retrofitted by FRP Subjected to Seismic Loads, International Review of Civil Engineering (I.RE.C.E.), 11(6), pp. 275–282.

[2] Qasim O A and Sultan H K 2020 Experimental Investigation of Effect of Steel Fiber on Concrete Construction Joints of Prism, IOP Conf. Series: Materials Science and Engineering, 745 (2020), doi:10.1088/1757-899X/745/1/012170.

[3] Sultan H K and Alyaseri I. 2020 Effects of elevated temperatures on mechanical properties of reactive powder concrete elements, Construction and Building Materials, 261 (2020) 120555. https://doi.org/10.1016/j.conbuildmat.2020.120555.

[4] Sultan H K Aziz H Y Maula B H Hasan A A and Hatem W A 2020 Evaluation of Contaminated Water Treatment on the Durability of Steel Piles, Advances in Civil Engineering, 2020, 6 pages, Article ID 1269563.

[5] New Sidney Lanier Bridge Pier Protection, Brunswick, GA. 2003.

[6] Buth C E Brackin M S Williams W F and Fry G T 2010 Guidelines for Designing Bridge Piers and Abutments for Vehicle Collision. Report Date: August 2010; Resubmitted: December 2010; Published: March.

[7] Jun L Hong W Xi-Yue H Nai-Shuai H and Ke L 2008 An FLIR Video Surveillance System to Avoid Bridge-Ship Collision, Proceedings of the World Congress on Engineering 2008 Vol. I.

[8] Gary R C David R C 2003 Nonlinear analysis of barge crush behavior and its relationship to impact resistant bridge design, Computers and Structures 81(2003) pp. 547–557.

[9] Junqing L Chaoyi X 2009 Vessel Collision Force Protection for Bridge Piers Determined According to Probability Method, Natural Science Foundation of China. 2009.

[10] Brown T G University of Calgary 2006 Confederation Bridge – An innovative approach to ice forces, the 2006 Annual Conference of the Transportation Association of Canada Charlottetown, Prince Edward Island.

[11] Aziz H Y and Jianlin M 2011 Design and analysis of bridge foundation with different codes, Journal of Civil Engineering and Construction Technology, 2(5), pp. 101-118.

[12] Aziz H Y and Abbass A. Abd-Noor 2016 Comparison of high speed railway bridge substructures, African Journal of Engineering Research, 4(2), pp. 36-42.

[13] AASHTO. Guide specification and commentary for vessel collision design of highway bridges. American Association of State Highway and Transportation Officials, 1994.

[14] AASHTO. LRFD bridge design specifications and commentary. American Association of State Highway and Transportation Officials, 2007.

[15] Republic of China Ministry of Railways. A Total of 200 Kilometer per Hour Passenger Railway Interim Design Provisions Code. 2007. (in Chinese).

[16] Aziz H Y Yun Y H E and Maula B H 2017 Dynamic Response of Bridge-Ship Collision Considering Pile-Soil Interaction, Civil Engineering Journal, 3(10).
[17] Aziz H Y Ma J L 2011 Study the Design and Analysis of Al-Shuhadah Bridge and Analysis by Using Plaxis Program of Finite Elements, *Advanced Materials Research, 243*, pp. 3676-3684.
[18] Aziz H Y Jianlin M Mohammed A A and Ni L 2011 Design of Substructure Bridge with Different Codes and Analysis Manually and by Using Plaxis Program 3D. *Third International Conference on Transportation Engineering (ICTE)*, pp. 1517-1523.
[19] Aziz H Y 2016 Using of Stones in Building the Foundations, *Muthanna Journal of Engineering and Technology (MJET), 4*(1), 2016.