УДК 624.014

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STUDY ABOUT WELDED STEEL ARCH BRIDGES FOR THREE TYPES , CALCULATIONS THE DEAD LOAD OF STEEL STRUCTURES FOR WELDED STEEL ARCH BRIDGES BY USE VALUE OF LIVE LOAD STUDY ABOUT WELDED STEEL ARCH BRIDGES FOR THREE TYPES,

Calculations the dead load of steel structures for welded steel arch bridges by use value of live load for three types welded steel arch bridges and this study applicable with max span length 130m.

Keywords: Analysis , type of steel arch (Rip), girders (Tie), shape of cross section, bridge floors, arches, wall girders, strength, resistance, stress stain behavior, optimal height of girder, analytical and numerical models, cost effectiveness indicators.

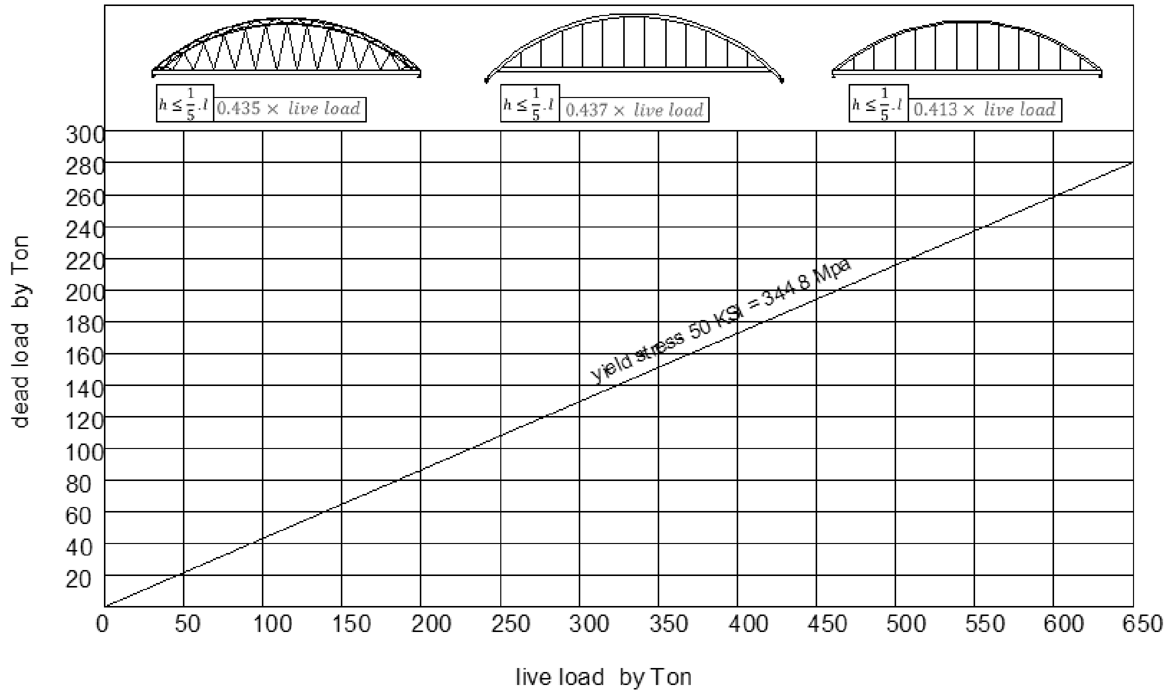
Introduction

The all designers of bridges need the value of dead load to start in the design structures of bridge by computer analysis program or by manual calculations

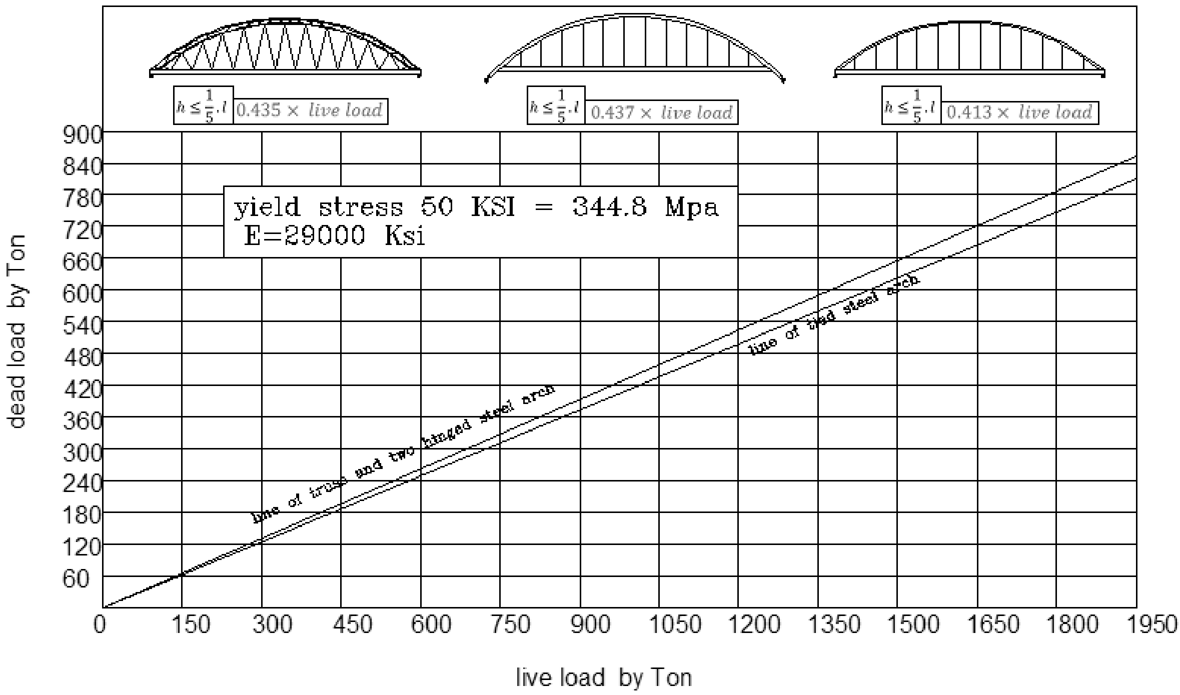
And some managers of constructions company need this value to determine the weight of steel structures the used in bridge to do the approximately prices or cost of constructions and also can use this value the student in academy studies in engineering colleges ,

So that we products some formulas to solve this problems where we obtained on three formula for three types steel arch as showing in fig(1) and fig(2).

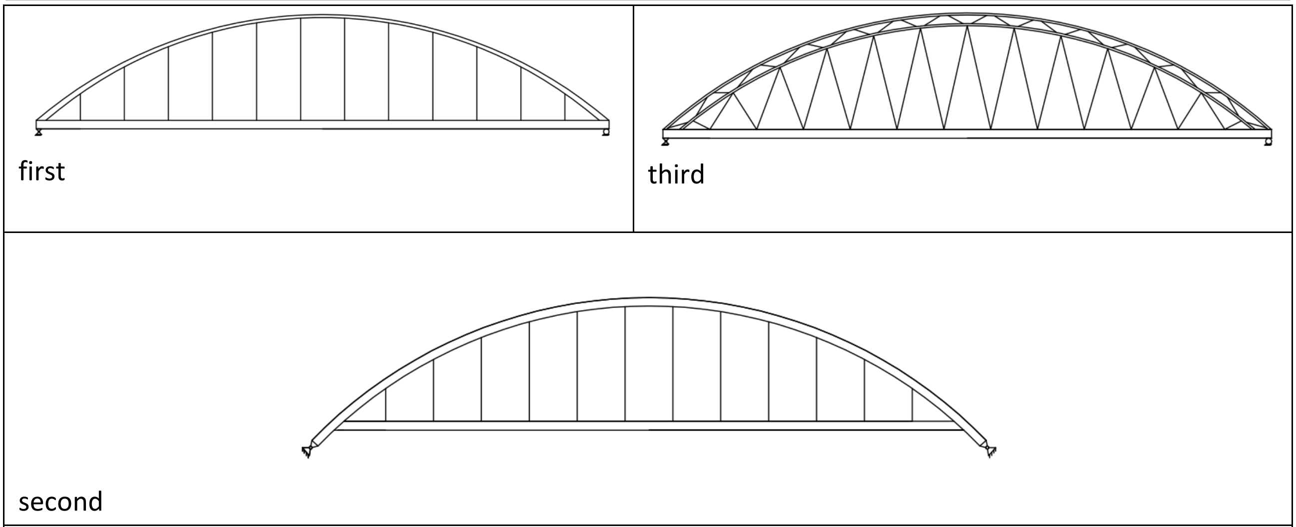
The two chart above includes two type chart the first in fig(1) Chart displaying value of dead load for live load with max load 650 ton can use with predestined bridges and the second type fig(2) Chart displaying value of dead load for live load with max load 1950 ton can use with vehicles bridges, the types of bridges are the first tied arch bridge (pin with roller) and the second type two pin and the third type truss rip (pin with roller) as showing in fig(3)



Fig(1) Chart displaying value of dead load for live load with max load 650 ton can use with predestined bridges



Fig(2) Chart displaying value of dead load for live load with max load 1950 ton can use with vehicles bridges



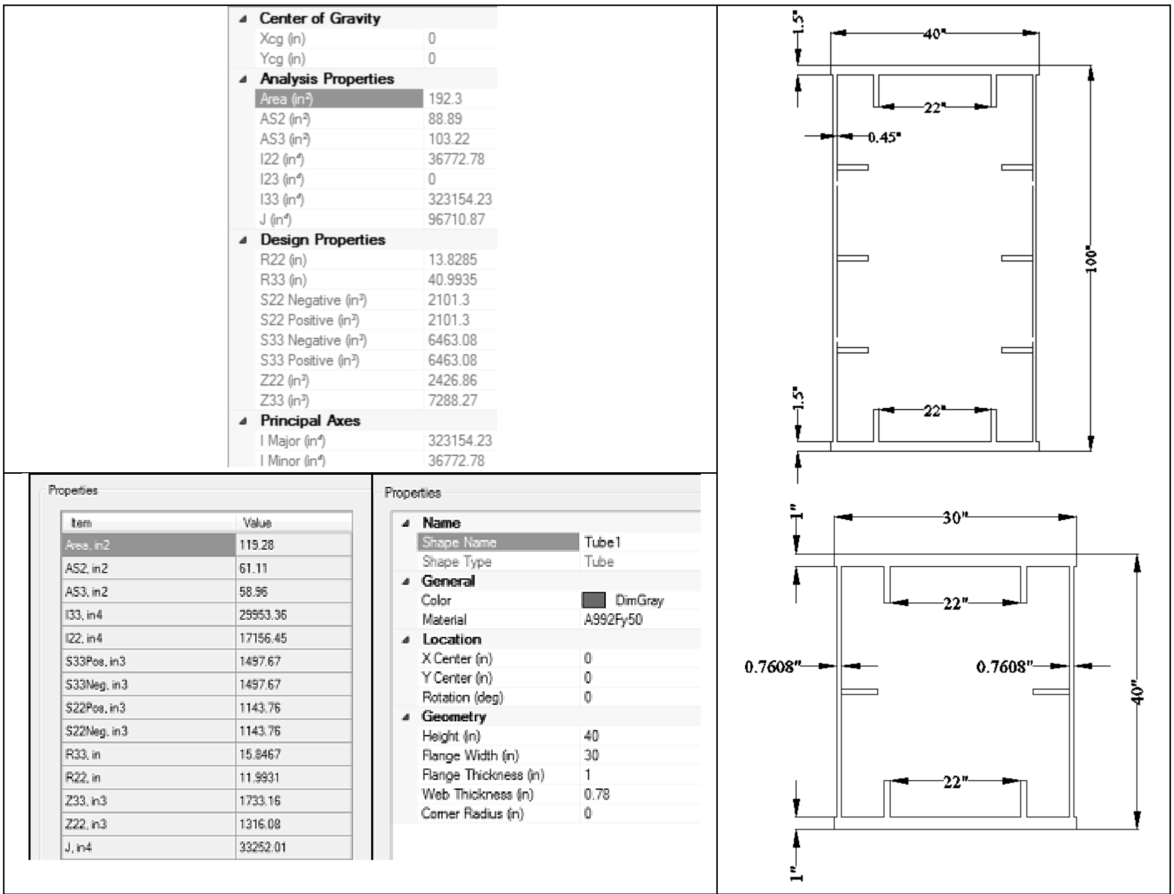
Fig(3) types of steel arch bridges span 130 m (425 ft).

In this study we use three example for three types as showing in fig(3) to determine this value and also we use the same materials ASTM A992 and the same length of span and the same loads the Numerical studies of arched steel bridge transitions on the example of the arch structure with a span of 130 m have been performed. Numerical studies of the examined structures were carried out using the ETABS 2015 software . the study is to determine the effect of a static constructive scheme of an arched bridge transition when using arched constructions with flexible arches and a rigid beam of a pedestrian or carriageway. For calculation, the same loading and dynamic factors are taken for all variants, which are included in the total load per running meter of the beam of rigidity of the arch bridge.

It consists of two arches, each of which has one hinged fixed support, and the other is hinged-movable. The function of tightening the arch is performed by the main girder of the carriageway of the girder. The girder (Tie) of the roadway and the arches are made of steel welded box-shaped profiles fig(4). Such a constructive solution is more mobile, but the effect of temperature influences is leveled by the moving support, which is important in conditions of a significant temperature difference.

The second variant is a two-hinged or pin arch structure, consisting of two arches, each of which has two pivot pivots. The main girder (Tie) of the roadway and the arches (Rip) are made of steel welded box-shaped profiles that serve as a puff. Such a constructive solution is more stringent, but the effect of temperature effects is more evident.

The third option is a steel structure .included from (Tie) steel box girder and steel arch (Rip) truss, and the truss included bottom and top truss and diagonal its made from steel box section, this option through the analysis by ETABS 2015 .we determine concentrations stress in the two end of arch where the end of the arch joins by welding with girder (Tie) because the transfer temperature strain from (Tie) and (Rip) to joint of connection so that products high stress concentration in this area and to reduce this value mast be increced the cross section area see fig(4) and moment of ineratia to lower the strssses in the end of the (Tie).



Fig(4) the span steel box girder for steel arch (Tie) with section of steel arch (Rip)

Numerical studies of the stressed-deformed state of steel arch bridge structures with a span of 130 m are carried out, with the same uniformly distributed loading 2.8 t/m^2 taking into account its on weight and the dynamic factor for the mobile load K_{Dinam}=1,15

CONDITIONS

The following are the Climatic Conditions:

Minimum Daily Average (January) 3.4 ℃

Extremely Lowest Temperature(January) -2 ℃

Maximum Daily Average (July) 43 ℃

Extremely Highest Temperature(July)55 ℃

Max metal temp by sun shine	80°C
Prevailing direction Wind	NW-SE
Max. wind speed	44.4 m/s (3 seconds)
Annual Rainfall	170mm
Max Relative Humidity	80%
Min Relative Humidity	25%
Seismic zone	Zone=2B (0.2g)
Soil Bearing Capacity	100Kpa

In fact this study is useful for Iraq or or hot areas in Middle east, For all the calculated options, the temperature effect of Iraq's conditions was accepted at a temperature drop during the operation of steel structures in the range of 60 ° C in south of iraq (at a minimum temperature of 5 ° C, the installation temperature was assumed to be 10-20 ° C).

Methodology

For the purposes of this paper, the nonlinear finite element analysis program ETABS 2015.has been used sufficient number of straight hermit beam finite element has been used for the analysis of the three option as showing in fig(3) accounting for the effect of both geometric and material nonlinearities. And by depending on the formula of EC3 [1] to determine the depth of welded plate girder we find some difference in the max moment in girder (Tie) and the steel arch (RIP) this difference affected on the value of max deflection of structure at center of span $130/2$ meter approximately equal $L/600$ and the value of deflection must be equals in three option to determined the end weight to total of steel structures .

$$h_0 = \sqrt[3]{k_{q1}^2}\sqrt[3]{k_R W_{x0} \lambda_\omega} \qquad ......(1)$$

1- $h_0 = depth\ of\ steel\ girder\ the\ box\ section\ by\ inches$
2-$k_R =\ design\ factor\ of\ box\ girder$
3-$\lambda_w = slendress\ ratio\ (depth\ ratio\ to\ web\ thickness)\ and\ depending\ on$ $the\ type\ of\ stiffners\ longtudunal\ or\ transfer\ as\ sowing\ in\ table$ 1

Table (1) Minimum Girder Web Thickness To Depth Ratios (AASHTO)

	A-7, A-373 & A-36 Steels	A.441 Steel 46.000 psi yield	A.441 Steel 50.000 psi yield
No stiffeners	**1/60**	**1/52**	**1/50**
Intermediate transverse stiffeners	**1/170**	**1/145**	**1/140**
Longitudinal stiffeners	**1/340**	**1/290**	**1/280**

And we also determine the minimum web thickness however the table (1) and this depending on the type of stiffeners and in this field we use box section longitudinal stiffeners with transfer stiffeners as showing in table (1)

$$MIN\ t_w = \frac{1}{280} \times 100 = 0.357'' \leq 0.450''\ OK\ thicknes\ of\ web\ for\ TIE$$

But to determine the max thickness we also we can use the following formula by AASHTO standard .

$$k = 5.17\left(\frac{D}{d_s}\right)^2$$

Where D = depth of steel box section.

d_s= distance between longitudinal stiffeners.

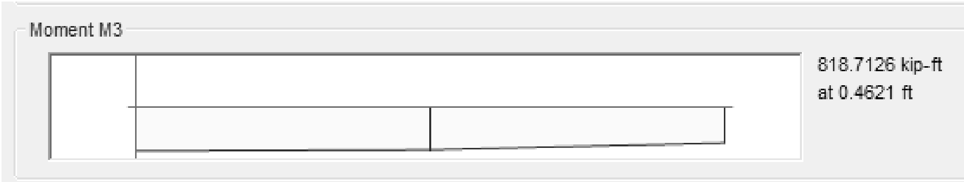
$$k = 5.17\left(\frac{100}{100/4}\right)^2 = 82.7$$

$$MAX\ t_w = \frac{D\sqrt{f_b}}{128\sqrt{k}} = \frac{100\sqrt{0.55 \times 50}}{128\sqrt{82.72}} = 0.450''$$
$$\geq 0.450''\ OK\ thicknes\ of\ web\ for\ TIE$$

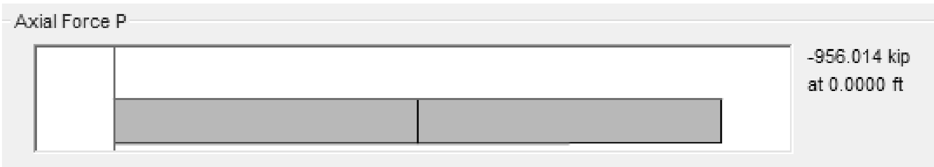
And from analysis program we obtained on the max moment from unsymmetrical load and max axial force from symmetrical load on surface of bridge . As showing below.

fig(5) Max moment in TIE unsymmetrical

fig(6) Max force in TIE unsymmetrical



fig(7) Max moment in RIP symmetrical



fig(8) Max force in RIP unsymmetrical

Above in fig(5) and fig(6) and fig(7) and fig (8) the max value for analysis steel structures of steel arch bridge the first option and the second and the third it is similar methods analysis and large number of geometrically and materially nonlinear analysis, and the including initial imperfection was carried out in order to capture the real highly nonlinear behavior of the arches and estimate their strength capacity. The result of this analysis were used in order to propose, an moment forces ,which need not be situated at the same cross section , obtained from a linear analysis. see table (3) Results of calculation of steel arch structures.

And by check the value of critical stress for (rip) and (Tie) section and by depending on the max value as showing above from max moment bending or max forces or max torque or max value to defalcation from unsymmetrical position and symmetrical we can find the critical stress .

And we do the combined compression and bending moment that is provided in EC3 for cross section by

$$\frac{N_{Ed}}{Xy.\frac{Af_y}{cM1}} + kyy\frac{M_{y.Ed}}{\frac{W_{pl.y}.f_y}{cM1}} \# z ..... \quad (2)$$

For the application of this equation for the case of arches $N_{Ed}, M_{y.Ed}$ are taken as the design values of compression and bending actions, which are the maximum values of the internal actions appearing along the arch, not necessarily at the same cross-section

Where A is the area of the cross-section, $w_{pl.y}$ is the plastic section modulus, f_y is the yield stress, x_y is a reduction factor due to in-plane flexural buckling, $k_{yy}$is an interaction factor due to combined compression and bending. The expression on the left part of inequality, defined as the utilization factor, should be bounded for design purposes by a maximum allowable value, denoted withφ, the load facter effected slightly on the properties of cross section for rip and the selendriens rotia $\lambda_\omega \approx h_0 / t_\omega$ ~~that~~ is approxmatly equal depth of section over thaickness of the web $\overline{\lambda}_{u\omega} \geq \overline{\lambda}_\omega \approx \lambda_\omega \sqrt{\frac{R_y}{E}}$ and in generaly in liner or non liner intrecation equal yirld stress over modulas of elastisity ander root squer for the materail of rip it use fo steel arch . And for welded plate girder (Tie) can teak moment bending at any distence along the length of span 130m to determaine the properties of cross section for symmetrical forcers over bridgeand as the fowlloing.

$$M_{xz} = M_{x0}\,[1-(\frac{2z}{l})^2] \quad ....(3)$$

Were is the $M_{x0} = ql^2/8$ moment bemding and I_{x0} moment of ineratia and W_{x0} the section modulas

$$I_{x0} = 2h_0^3 t_\omega / 12 + 2A_f h_0^2 / 4 + 2b_f t_f^3 / 12\,; \qquad .....(4)$$

$$W_{x0} = \frac{2I_{x0}}{h_0} = 2h_0 t_\omega / 6 + A_f h_0 + 2b_f t_f^3 / (6h_0)\,; \qquad ......(5)$$

$$W_{x0} \approx W_{xf} + W_{x\omega}; \qquad W_{x\omega} = 2h_0^2 t_\omega / 6; \qquad W_{xf} = A_f h_0 + 2b_f t_f^3 / (6h_0)\,, \quad .....(6)$$

$$c_f = \frac{M_{xf}}{M_x} = \frac{A_f h_0}{2h_0^2 t_\omega / 6 + A_f h_0} = \frac{1}{2h_0 t_\omega / (6A_f) + 1} \qquad .....(7)$$

$$c_f = \frac{M_{xf}}{M_x} = \frac{A_f h_0}{2h_0^2 t_\omega / 6 + A_f h_0} = \frac{A_f h_0}{W_x} \rightarrow A_f h_0 = c_f W_x \quad .....(8)$$

And the W_{xf} section modules of flanges in [6] and $W_{x\omega}$ section modules of the web andt_ω thickness of web and $A_f = b_f t_f$ area of flanges for the welded section

And the value coefficient of k_{q1}in formula [1] depending on the fowling table (2) to determine the depth of cross section .

table (2)

Coefficient k_{q1}

$\frac{h_0 t_\omega}{A_f}$	$\frac{1}{(h_0 t_\omega / (3A_f) + 1)}$	ψ_p **=1,05 ;** $\psi_{p\omega}$**=1,2**		ψ_p **=1,05 ;** $\psi_{p\omega}$**=1,1**	
		k_{q1}	$\sqrt[3]{k_{q1}^2}$	k_{q1}	$\sqrt[3]{k_{q1}^2}$

0,5	0,857143	0,8660	0,9086	0,9045	0,9353
1	0,75	0,8101	0,8690	1,1969	1,1273
1,25	0,705882	0,7859	0,8516	1,1611	1,1047
1,5	0,666667	0,7638	0,8355	1,1284	1,0839
1,75	0,631579	0,7434	0,8206	1,0983	1,0645
2	0,6	0,7246	0,8067	1,0705	1,0465
2,5	0,545455	0,6908	0,7815	1,0207	1,0138

And the $k_{q1} = \sqrt{\dfrac{\psi_{pf}}{\psi_{p\omega}(h_0 t_\omega / (3A_f) + 1)}}$ ..... (9)

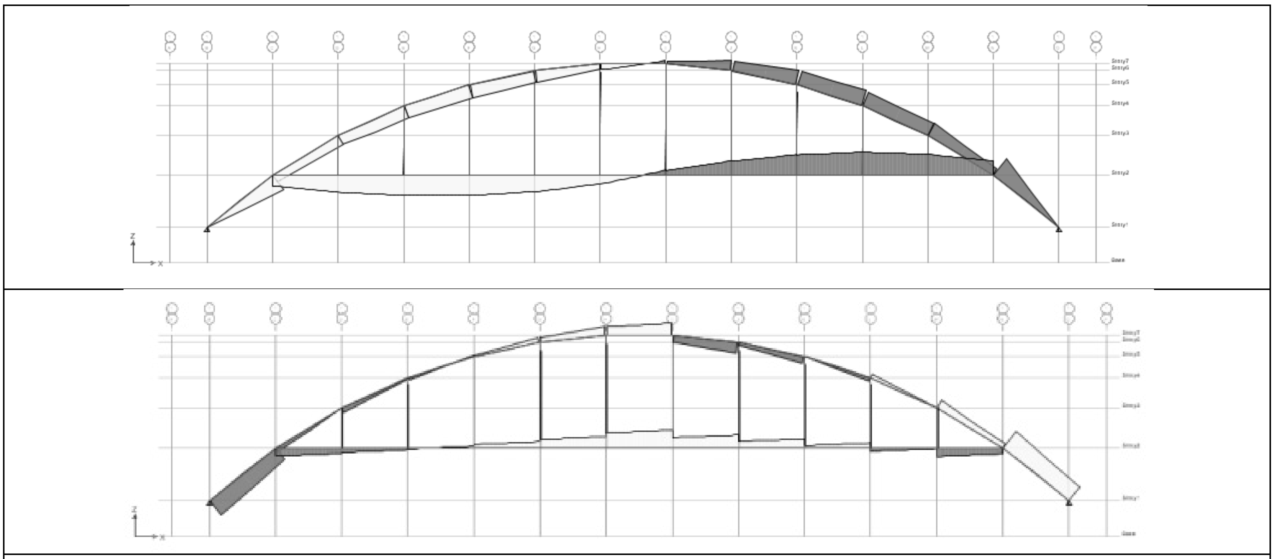
ψ_{pf} it is constructive coefficient ratio , $\psi_{pf} = \dfrac{m_{f0} + \sum m_{irf}}{m_{f0}}$.

$\psi_{p\omega}$ it is constructive thickness of wall coefficient ratio, $\psi_{p\omega} = \dfrac{m_{\omega 0} + \sum m_{ir\omega}}{m_{\omega 0}}$.

m_{f0} weight the welded plate girder with out any stiffener longitudinal or transfer .

$\sum m_{irf}$ the total of weights of additional all parts and all type of stiffener.

The qualitative characteristic of the displacements and the bending moments and longitudinal forces is shown in Fig.(9) under symmetrical and unsymmetrical loading to avoid option number second in fig(3).



Fig(9) The results of calculating the arched combined bridge design with two pin hinged supports with unsymmetrical and symmetrecal loading, the maximum values of bending moments and and the longitudinal forces in the bar of cruelty in the arch.

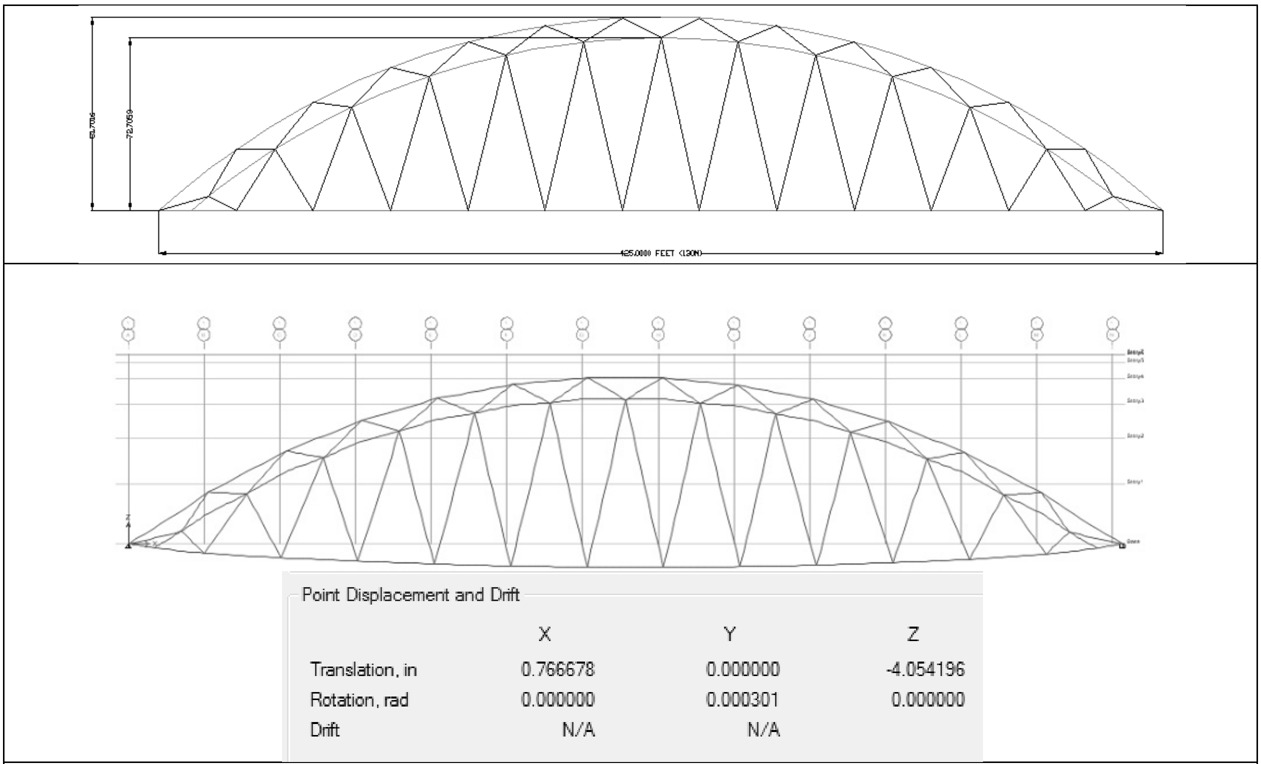


Fig.10 General scheme of the arch combined structure of a metal bridge with an arched farm span of 130 m. General scheme of displacements under symmetric loading

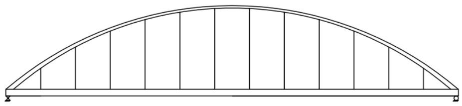
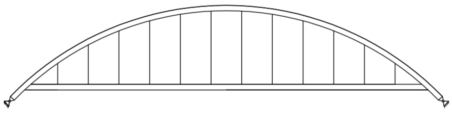
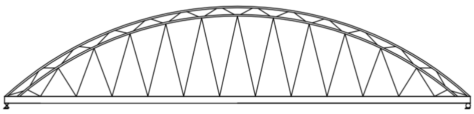
However, in the plate girder, the web is part of a built-up member. When the critical buckling stress in the web is reached, the girder does not collapse. The flange plates carry all of the bending moment, the buckled web serves as a tension diagonal, and the transverse stiffeners become the vertical compression members.

This has the effect of making the girder act as a truss n Fig. 10 shows the general scheme of the steel arch truss third option as showing in fig(3) combined structure of a metal bridge with an arch truss of 130 m. Also, the results of the static calculation: the general scheme of displacements under symmetrical loading and maximum displacements, which amounted to 4.05 inches = 10.287 cm. Numerical studies of the examined structures were carried out using the ETABS 2015 software program . the fowling all the maximum value for analysis the steel arch bridge in fig(3) and for the three option

The steel consumption on the arched structure based on the calculations made (the conventional theoretical mass of the steel arches, beams of stiffness, diagonal elements of the truss, connections between the trusses, without additional metal structures for the foundation of the pavement) amounted to 268.5 ton , including: a bar of rigidity - 85 tons, an arch through construction 96 tons, diagonals (lattice of the truss) and connections between the arches - 57.3 + 30.2 = 87.5 tons.

table (3)

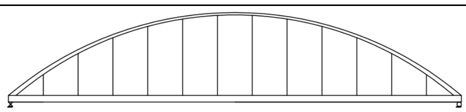
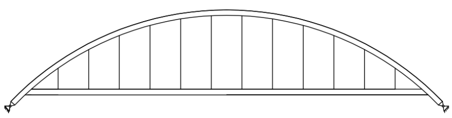
Results of calculation of steel arch structures

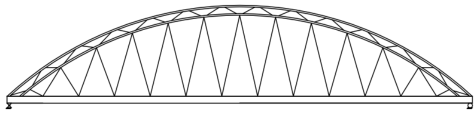
Тип арочной стальной конструкции моста	structure	Max (M) Kip.ft	Max (N) Kip	weight (ton)	total weight (ton)	Max deflection (in)
 option (1) pin with roller	arch (Rip)	818	-1495	138.9	282.93	-9.4"
	girder (Tie)	-8611	+1184	126.6		
	cable	29.6	+174	17.42		
 option (2) pin with pin	arch (Rip)	3455	1100	176.6	284.27	-9.6"
	girder (Tie)	5471	154	90.25		
	cable	-27.6	170	17.42		
 option (3) pin with roller	arch	313.5	-1027	96	268.5	-2.6"
	girder (Tie)	1435.6	+1328.2	85		
	diagonal with cable	17	+117	57.3 + 30.2		

Comparison of specific weights of structural elements.

Table.4

Comparison of the specific weights of structural elements of bridges for three variants

option		Relative weight, %: The steel arch of the bridge (Rip) of the total theoretical weight of conditional	Relative weight, %: welded girder for (Tie) arch bridge on the total theoretical weight of conditional	Relative weight, %: tension cable on the total theoretical weight of conditional
1	 option (1) pin with roller	49%	44%	7%
2	 option (2) pin with pin	62%	33%	5%

3	 option (3) pin with roller	36%	31%	33%

The analysis of the specific weights of individual structural elements showed that in the welded plate girder (Tie) less systems, in which the rigidity bar performs and the hollow function of the arch tightening, the costs of steel on the arch were 36% -38%, and in the arch spacer system 62%. At the same time, specific The weight of the structure of the girder (Tie) and connections in the first and second versions is 62% -64%, and in the spacer system (option 2) - 38%.

The Strength and stability analysis of the spatial arch structure. There is no bracing between the two main arches. Flexible hangers carry the bridge floor system, and it is structurally independent of the main arches. A major concern of designer was to ensure sufficient safety margin with regard to out-of-plane stability of the arch structure.

The load-carrying system comprises main arches and the auxiliary half-arches interconnected by the cross bars. These elements, attached to the both sides of the bridge, significantly enhance the lateral stiffness of the whole structure. Stability of such a structure cannot be checked by simple analytical formulas.

Table.5

Different thrust of horizontal and vertical forces between three type steel arch for one end supported

NO	type of steel arch	force X (Kip)	force Z (Kip)	percents %
1	tied solid web arch bridge 130m option 1	0	916.04 (416.3 ton)	92%
2	two hinged steel arch bridge 130m option 2	1202 (546.3 ton)	956.4 (434.7 ton)	100%
3	tied Truss steel arch bridge 130m option 3	0	917.5 (417 ton)	92%

Moreover, existing code-type methods do not allow consistent verifying of arch stability beyond the elastic range.

$\sum_{}(\mathit{live\ load})\ 50 \times 13 = 650\ \mathit{ton}$

A parameter (design factor) β is proposed to evaluate the technical solution of combined arched steel bridges. This parameter is the ratio of the weight of the bridge structures (one arch, one girder and cable or rip and tie and cable) to the total live load $\sum\_(live\ load)\ 50 \times 13 = 650\ ton$, which amounted to 650 tons per arch. The parameter β is calculated for each variant of the arch bridge structures.

Now we can obtained on the design factor for three types arch as the following :

$$\beta_{tied} = \frac{\sum_w tied}{\sum_{live\ load} tied} = \frac{282.93}{650} = 0.435\ time$$

$$\beta_{two\ hinged} = \frac{\sum_w two\ hinged}{\sum_{live\ load} two\ hinged} = \frac{284.27}{650} = 0.437\ time$$

$$\beta_{truss} = \frac{\sum_w truss}{\sum_{live\ load} truss} = \frac{268.5}{650} = 0.413\ time$$

The final we define (β) the design factors for save weight of steel arch :

Table.6

Design factors for save weight of steel arch $(\boldsymbol{\beta})$

type of load	Tied steel arch	Two hinged steel arch	Truss steel arch
Dead load β	$0.435 \times\ live\ load$	$0.437 \times\ live\ load$	$0.413 \times\ live\ load$

The dead load is obtained from the weight of the permanent parts of the bridge structure itself. This includes the steel deck, arches, wind bracing, columns and bracing under the deck. The dead load is calculated by obtaining the mass of the section of bridge being considered and multiplying it by the appropriate design The value above with out dynamics loads where some time the dynamics loads equals 15 % to 20% live loads . and this factor with out earth quack factor.

And from analysis we find only thrust horizontal very big in two side of arch two hinged type (2) yes the save weight 284.2 ton but the horizontal thrust very large in abutment where the need to more piles or reinforcement concrete to improve the resulting of reactions forces in abutments so that will be high expansive and No economy .

Conclusions

1- The result of our research was the obtaining of the parameter (weight) of the weight of the steel structures of the steel arch bridges, the ratio β for three types (variants) of steel combined arch bridges. The weight factor βG is adopted as the ratio between the theoretical weight of the bridge and the mobile load on the bridge arch bridge. It is established that this parameter of the weight of the steel structures of the arch bridge has a range of variation: β = 0.435 ... 0.437...0.413.

2-An important new scientific result of the research is the comparison of theoretical calculations and practical application of the method for determining the optimum height of box sections at the first stages of designing combined arch bridges with the use of welded steel girder profiles. These studies determined the range of coefficients that should be assigned when calculating the optimum beam height of a box section. For beams of stiffness = 1,2..1,3, = 1,1..1,2, = 1,25. For the arched part of the bridge, the values of the coefficients should be assigned in the range: = 1,1..1,3, = 1,4..1,6, = 1,25.

3- Specific weights of separate structural elements for each type of arch combined steel structure are established. For spacer systems, the costs of steel on the (Tie) of rigidity are 32-36%, and for the arch and communications, respectively, 68-64%. And for arch (Rip) (type spacer structures, the cost of steel on the arch is 68% and on the reinforcement bar 32%.

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АНОТАЦІЯ

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ДОСЛІДЖЕННЯ ЗВАРНИХ СТАЛЕВИХ АРКОВИХ МОСТІВ ТРЬОХ ТИПІВ. ВИЗНАЧЕННЯ ПОСТІЙНОГО НАВАНТАЖЕННЯ ВІД ВЛАСНОЇ ВАГИ СТАЛЕВИХ ЗВАРНИХ КОНСТРУКЦІЙ АРКОВИХ МОСТІВ ЧЕРЕЗ РУХОМЕ НАВАНТАЖЕННЯ

Проведені числові дослідження раціональних конструкцій сталевих арочний мостових конструкцій з метою визначення впливу умов обпирання арки та загальної схеми конструкції на загальні витрати сталі для умов Іраку.

Виконано числові дослідження аркових сталевих мостових конструкцій на прикладі арочної системи прольотом 130 м. Чисельні дослідження розглянутих конструкцій виконані за допомогою програмного комплексу ETABS-2015 та CSI bridge -2014. Метою досліджень є визначення впливу статичної конструктивної схеми аркового мостового переходу при застосуванні аркових конструкцій з гнучкими арками і жорсткої балкою пішохідної або проїзної частини. Для розрахунку прийняті для всіх варіантів однакові навантаження і коефіцієнти динамічності. Виконано порівняння 3 варіантів розрахункових схем аркових конструкцій мостових переходів, щонайбільш застосованих в умовах Іраку.

Перший варіант – це двохшарнірна аркова конструкція мостового переходу, що складається з двох арок, кожна з яких має одну шарнірно-нерухому опору, а іншу шарнірно-рухому. Функцію затяжки арки виконують головні балки проїзної частини мостового переходу. Як балка проїзної частини, так і арки виконані зі сталевих зварних коробчастих профілів.

Другий варіант – це двохшарнірна аркова конструкція, що складається з двох арок, кожна з яких, яка має дві опори шарнірно нерухомі. Головні балки проїзної частини та арки виконані зі сталевих зварних коробчастих профілів, які служать затяжкою.

Третій варіант (рис.9,в) – це сталева конструкція, в якій основними несучими елементами є дві шарнірно розташовані ферми арочного обрису. Нижній пояс ферм є одночасно і головними балками проїжджої частини. Ферма також має при установці на опорні частини мостового переходу одну шарнірно нерухому опору, та другу шарнірно рухому.

У конструктивному вирішенні прийнято, що конструкція гнучких арок виконується зварного коробчастого перерізу, а балка жорсткості також проектується як зварна коробчаста балка.

Результатами досліджень стало отримання параметра (фактора) ваги β_G сталевих конструкцій арочного моста для трьох типів (варіантів) сталевих комбінованих арочних мостів. Фактор ваги моста β_G прийнятий як співвідношення між умовною теоретичною вагою моста і рухомим навантаженням. Встановлено діапазон зміни фактору, що складає β_G = (0,437 ... 0,413). Отримані результати дозволяють спрогнозувати масу сталевих конструкцій пролітної будови моста залежно від сумарного корисного навантаження.

Ключові слова: пролітні будови мостів, арки, суцільностінчасті перерізи, міцність, стійкість, напружено-деформований стан, оптимальна висота перерізу, аналітична та числова моделі, показники економічної ефективності.