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Are Reinforced Concrete Girder Bridges More Economical Than Structural Steel Girder Bridges? A South African Perspective

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ABSTRACT

This study investigated the cost-effectiveness of steel girders compared with conventional reinforced concrete girders used in bridge construction in South Africa. The investigation considered an existing bridge which required widening as a result of increased traffic flow. The consulting engineers chose two 10 m reinforced concrete girders for the end spans, while two 22 m post-tensioned reinforced concrete girders were used for the central spans. To determine the cost-effectiveness of the consulting engineer's choice, steel girders were designed for the 10 m and 22 m beams based on the relevant South African design codes of practice. The analysis was conducted for both non-composite and composite action between the steel girders and the reinforced concrete bridge deck. Based on the design calculations, a cost comparison was performed. The investigation revealed that steel girders are an economically viable option when off-the-shelf steel sections up to 10 m in length are considered. For the 10 m girders, a significant cost saving was achieved for both non-composite and composite construction compared with reinforced concrete. However, for the 22 m spans, the post-tensioned reinforced concrete girders are significantly more cost-effective compared with steel plate girders. Thus, bridge design engineers should consider steel girders as an option during the conceptual design stage for end spans. A survey was also conducted among senior engineering professionals to determine the reason(s) for the apathy towards using structural steel sections as girders in bridge construction in South Africa.

KEYWORDS: Composite bridges, RC girders, Steel girders, South Africa, Cost analysis.

INTRODUCTION

The increase in housing developments in Kuils River in the Western Cape Province of South Africa led to a significant increase in traffic volumes during peak hours at the western end of Bottelary Road (M31). This necessitated the widening of this section of Bottelary Road, including the bridge that crosses over the Kuils River Freeway (R300). The existing bridge's end spans consisted of two 10 m simply supported reinforced concrete (RC) girders while the central spans were supported by two 22 m simply supported post-tensioned RC girders spanning the dual carriageways over the Kuils River Freeway. Figure 1 shows a front elevation of the bridge with its two main and two end spans.

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Types of Bridge

Bridges are composed of various elements to transfer traffic and dead loads from the bridge deck to the substructure. A typical freeway bridge, as depicted in Figure 1, is composed of a bridge slab which transfers the dead and traffic loads to the simply supported girders. These loads are then transferred to the piers and finally to the subsurface through the foundation. All of these elements are generally constructed of RC. The bridge girders will deflect due to the applied load, causing the region below the neutral axis of the cross-section to be in tension while the remaining cross section is in compression. Therefore, the material used must resist the design loads and also be cost-effective. RC and structural steel sections are the only viable materials, depending on the bridge span. Table 1 lists the longest bridges in the world as a function of bridge type, span, year of construction and the material used for the girders (Virola, 2006).

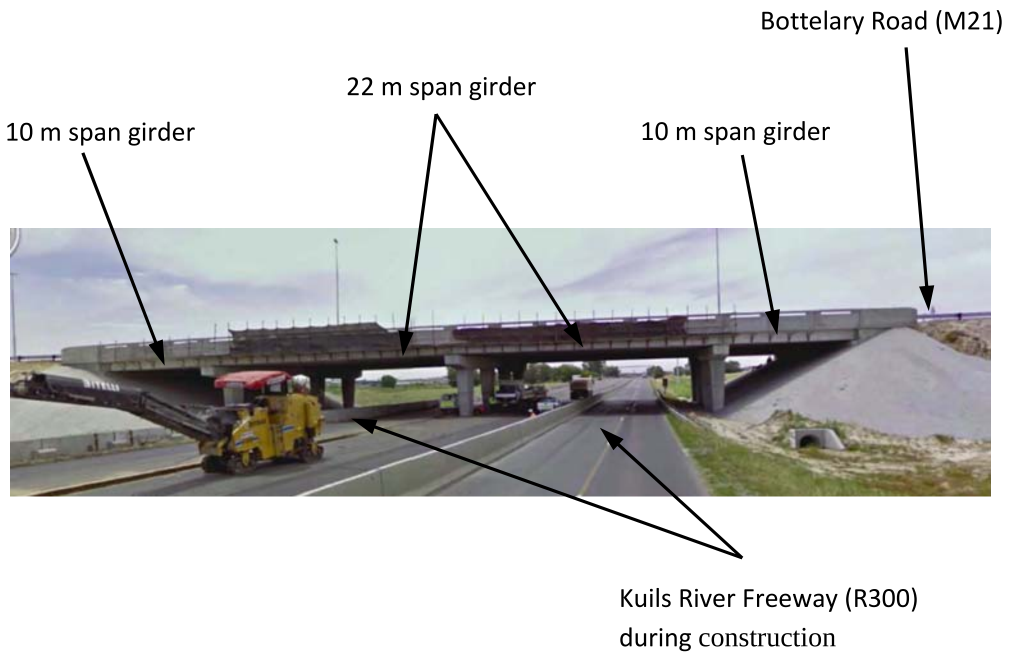


Figure (1): Widening of the M21 and the R300 during construction

Table 1. Longest bridge spans in terms of type and material used

Name of bridge	Type of bridge	Year of completion	Central span (m)	Material used for the girder	Country
Akashi-Kaiyo	Suspension	April, 1998	1991	Steel	Japan
Russky	Cable-stayed	July, 2012	1104	Steel	Russia
Chaotianmen	Arch	April, 2009	552	Steel	China
Pont de Quebec	Truss girder	December, 1919	549	Steel	Canada
Wanxian	Arch	July, 1997	425	RC	China
Shibanpo	Prestressed girder	July, 2006	330	RC	China
Ponte Costa e Silva	Box/Plate girder	March, 1974	300	Steel	Brazil

From Table 1, it is evident that structural steel is preferred for bridges with spans greater than 500 m. This is due to its superior strength-to-weight ratio compared with RC. However, for bridges with shorter spans, up to 330 m, either pre / post tensioned concrete or structural steel can be used.

The tendency in South Africa is to use RC girders in freeway bridge construction. This is validated in that only 3 of the 104 freeway bridges in the Western Cape are constructed using steel girders (Viljoen, 2012). A similar trend is also observed in the other 8 provinces in South Africa (Kruger, 2012). The span dimensions of these bridges are similar to those of the Bottelary Road bridge. From this statistic, it is clear that RC is preferred above structural steel for the construction of bridge girders in South Africa. No sound basis was found for this decision. This approach is opposite to that of countries such as the USA, which uses predominantly steel girders in freeway bridge construction. A significant limitation to using steel girders in freeway bridge construction is the largest available off-the-shelf I-section in South Africa. The largest commercially available off-the-shelf I-section in South Africa has an overall height of 510 mm, a flange width of 210 mm with a mass of 122 kg/m. The cross-section has a second moment of inertia of 762 x 106 $mm^4$and an elastic section modulus of 2 798 x 103 mm^3.

This abnormality in material usage leads to question why RC is preferred for bridge girder construction in South Africa. The main factors that influence the material choice for bridge girders are:

- Longest span of the bridge;
- Location;
- Experience of the bridge engineer working with structural steel;
- Cost of the structural steel and availability of larger off-the-shelf steel sections;
- Crane capacity required and availability of these cranes;
- Time constraints;
- Safety requirements;
- Standardized RC sections used by provincial road agencies and consulting engineers;
- RC is the norm.

To answer This question, this investigation considered the responses from a survey among bridge design professionals, as well as designing steel girders to perform a cost analysis. The purpose of the survey was to determine the reason(s) for the apathy amongst bridge designers towards using structural steel sections as bridge girders. The results clearly show that there are no convincing reasons why RC should remain the preferred material for bridge girders. Another focus of the investigation was to determine whether steel girders are more cost-effective than traditional RC girders. This was established by designing steel girders with the concrete bridge deck for either composite or non-composite action. Various corrosion treatment options were also considered during the cost analysis.

The aim of this paper was to question the validity of assuming that RC bridge girders are more economical than structural steel sections and to elicit debate on this topic. The paper also shows that off-the-shelf I-sections can be used for spans up to 10 m without cambering. The impact of this finding could result in cheaper bridges being constructed if the recommendations are implemented. Steel girders for freeway bridge construction could be constructed off site in modular format and be easily assembled on site. This could be especially viable in rural and semi-rural areas where skilled labour is scarce and it would be expensive to gather a team of artisans to construct a bridge for passenger vehicles. The same would also apply to urban environments and, in addition, off-site assembly would be less disruptive to traffic during the construction phase.

SURVEY

A survey questionnaire was developed to determine the reason(s) why steel girders are seldom used in freeway bridge construction. The questionnaire was sent to numerous companies that specialize in freeway bridge construction.

The questionnaire required only experienced bridge or structural engineering professionals to participate in the survey. The survey yielded 10 responses, with an average of 16years of bridge design experience. The authors felt confident that the years of experience meant that only senior engineering professionals completed the questionnaire. Since not many consultants specialize in bridge design, the authors believe that the responses would be similar if a wider audience were approached.

The questionnaire was designed as a Web-based survey with the respondents being required to indicate their level of agreement with various statements and, where necessary, to comment. The important results of the survey are now presented. Figure 2 shows the responses obtained from the survey. Since only ten responses were obtained, the average level of agreement can be considered as the percentage.

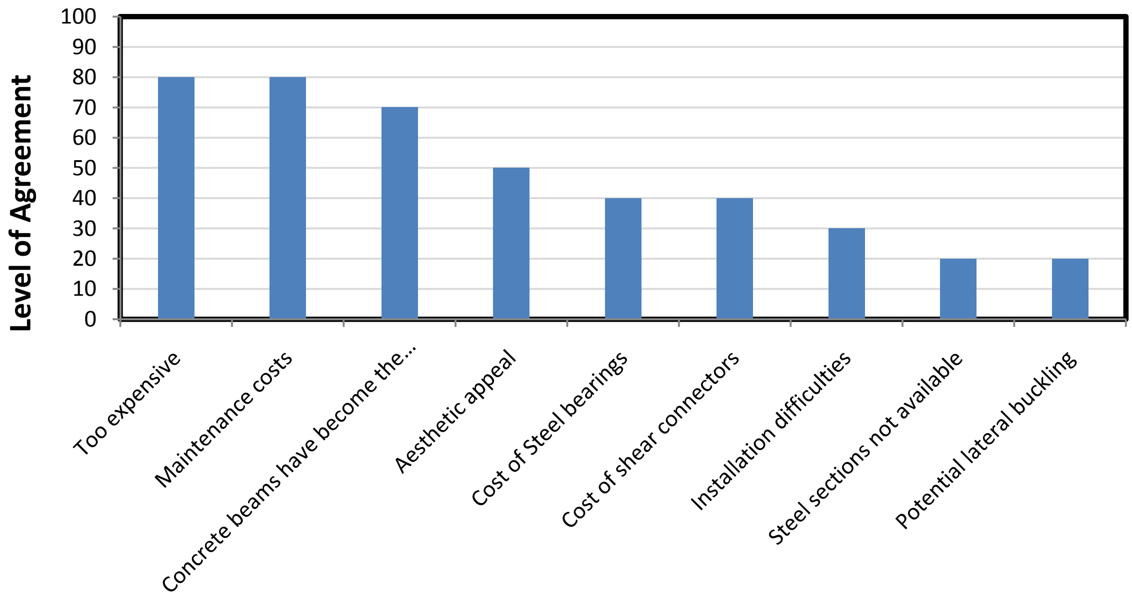
From the survey, it is evident that the most important reasons toward the apathy of using steel girders are:

- That steel girders are too expensive irrespective of the span considered in freeway bridges.
- That extremely high maintenance costs make structural steel girders an uneconomical option.
- That RC girders have become the norm based on the designers familiarity with RC and also to ensure uniformity with other bridges in the vicinity.
- That the aesthetic appeal of RC girders is more attractive compared with structural steel.

Other important findings from the survey to which the respondents were simply required to answer with either ‘Yes’ or ‘No’ are that:

- 70% agreed that there is an opportunity for modular bridge construction in South Africa.
- 60% did not feel comfortable designing steel structures using weathering steel in coastal areas.
- 70% thought that composite bridges would *not* become popular, even though the price of steel has decreased since mid-2008.

Based on the survey findings, it is evident that the apathy towards using steel girders is prejudge mental and not substantiated by a regular cost analysis.



Factors affecting composite bridge construction in South Africa

Figure (2): Responses from the survey

Price Index of Structural Steel

The authors assume that the statement with respect to steel sections being expensive was based on the situation at a specific period of time and not on a continual basis. Figure 3 shows the fluctuation in the price index of I- and H-sections per ton since January 2005 (Erling, 2012). The price index can be assumed to be equivalent to the manufactured cost of structural steel.

Figure 3 shows that the price index of steel increased significantly from $1 011/ton in January 2008 to $1450/ton in November 2008. This amounts to a 43% increase within 10 months. In February 2011, the price index of steel decreased to its lowest level of $925/ton after the significant increase in November 2008. Since September 2011, the price index of steel has stabilized to approximately $1 150/ton. This proves that the price of steel fluctuates considerably and therefore a cost analysis is important at the specific time to provide the client with the most affordable project cost.

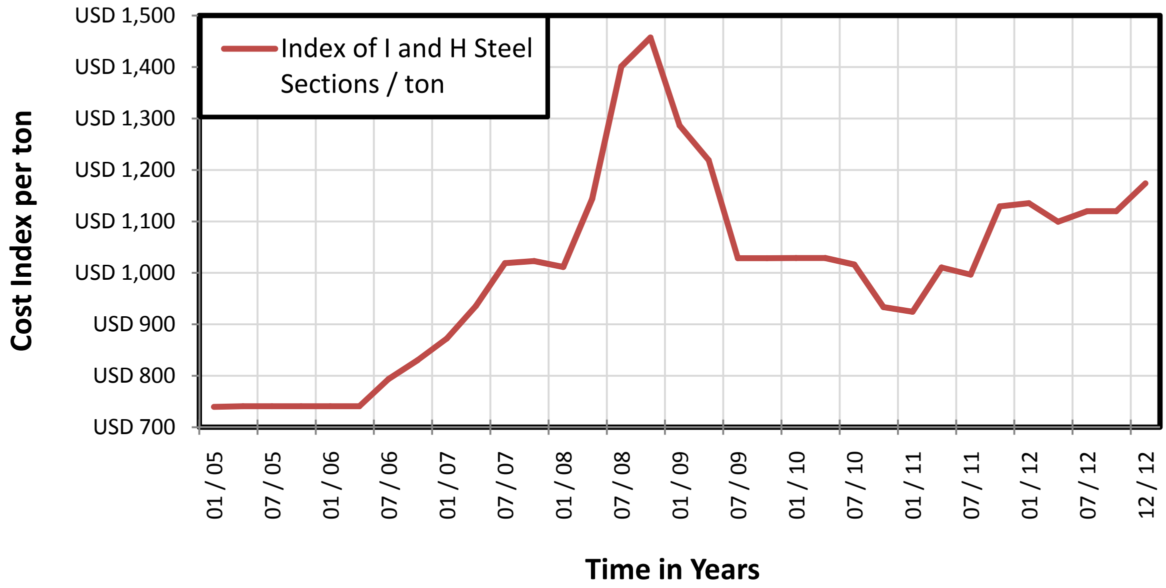


Figure (3): Cost index of steel per ton since January 2005

BRIDGE LOADING

Bridge Details

The existing bridge consisted of two 22 m simply supported central spans and two 10 m end spans which were 8.6 m wide. This width allowed for a single carriageway in both directions. The existing bridge deck was supported on 22 m post-tensioned and 10 m RC girders spaced 1 m apart. The consulting engineer used the same configuration (length and width) to conform with the existing bridge thereby ignoring the potential cost saving if steel girders were used (Smith, 2010).

Traffic Loading

Since the bridge is located in the vicinity of an industrial area, it was designed to accommodate both passenger and heavy industrial vehicle loading. The maximum traffic load which the bridge could be subjected to was determined based on the *Code of*

Practice for the Design of Highway Bridges and Culverts in South Africa (TMH 7, Parts 1 to 3,1989) for the following cases:

- NA loading – normal loading on the bridge.
- NB loading – abnormal loading on the bridge.
- NC loading – super loading on the bridge.

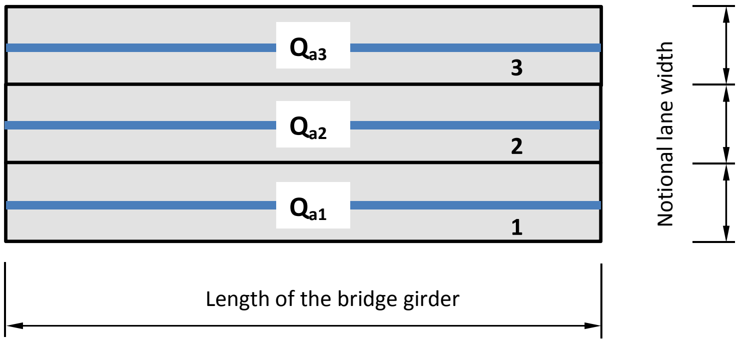


Figure (4): Effective length of each notional lane

NC loading is only required on selected routes. However, for this analysis, all 3 traffic loading conditions were considered. From the above loading conditions, the maximum shear force (SF) and bending moment (BM) were obtained for the 10 m and 22 m simply supported beams. Wind loads were omitted from this study as they are not significant in short- to medium-span bridges (Kayser and Nowak, 1989). Based on the code, no allowance was made for impact and dynamic effects. Three notional lanes were required for an 8.6 m carriageway width.

NA Loading

This section comprises of three sub-sections to determine the worst loading for type NA loading. For conditions 1 and 2, the loads are combined, while condition 3 is analyzed individually.

Condition 1

For condition 1, a nominal distributed load, Q_a, applied along the length of the beam is obtained as either;

- For an effective beam length (L_{eff}) < 36m, $Q_a = 36\ kN/m$
- For an effective beam length (L_{eff}) > 36m, $Q_a = \frac{180}{\sqrt{Leff}} + 6\ kN/m$

where L_{eff} is obtained from the sum of the effective length of the notational lane width shown in Figure 4 as:

For Q_{a1}, $L_{eff} = L_1$.

For Q_{a2}, $L_{eff} = L_1 + L_2$.

For Q_{a3}, $L_{eff} = L_1 + L_2 + L_3$.

Thus Q_a is obtained as the maximum of Q_{a1}, Q_{a2} and Q_{a3}.

The above load must also be distributed transversely across the notational lane width to represent two wheels at either end of the axle of the vehicle. The un-factored uniform distributed load, Q_a, is thus divided equally across each notional lane width as given in Equation 1.

$$Q = \frac{Qa}{Nominal\ Lane\ Width}\ kN/m. \quad (1)$$

Condition 2

For condition 2, a nominal axle point load of $\frac{144}{\sqrt{n}}$ kN should be applied on the notional lane. The factor, *n*, is the loading sequence number of the relevant lane. Thus, the worst case scenario, that is *n* = 1, results in a point load of 144 kN. As with condition 1, the point load must also be distributed transversely across the lane width. This load must be positioned on the beam to yield the maximum SF and maximum BM. Figure 5 shows the load arrangements to obtain the maximum SF and maximum BM of the un-factored traffic for the combined case.

Condition 3

For this condition, a nominal load of 100 kN is applied acting on an area of 0.1m^2, anywhere within the carriageway to obtain the maximum SF and maximum BM. The load arrangement is the same as for Figure 5, except with the uniform distributed loading removed.

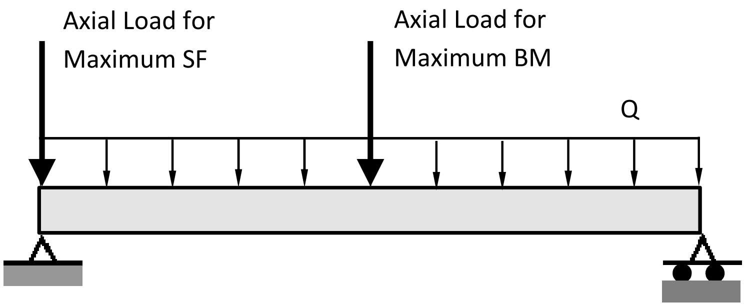


Figure (5): NA loading configuration for conditions 1 and 2

NB Loading

This load condition represents a single abnormal heavy loaded vehicle travelling over the bridge. This load condition comprises of two load magnitudes, NB 24 and NB 36. NB 24 load condition has a 60 kN wheel magnitude resulting in a load of 240 kN per axle, while NB 36 load condition has a 90 kN wheel magnitude resulting in a load of 360 kN per axle. Both load conditions are applied in the same manner. NB 36 loading condition was assumed, since the bridge is located in an industrial area and could be subjected to abnormal loading.

Figure 6 shows the vehicle's wheel load distribution within a nominal lane width. Each wheel distributes a load of 90 kN acting across a 1 m strip width as shown in Figure 6. The maximum SF and BM can be obtained using influence lines or a conventional approach by moving the point loads along the span length.

The spacing, "**a**", between the inner wheels of the vehicle varies in increments of 5 m starting at "**a**"= 6m.

NC Loading

The load obtained from this condition is to simulate multi-wheeled trailer combinations with controlled hydraulic suspension intended to transport very heavy indivisible payloads. Figure 7 shows the load arrangements of the Standard Type NC-30 x 5 x 40 loading defining variables "**a**" and "**c**". For this loading condition, there is no notational lane and the width of dimension "**b**" is taken as the minimum of 5 m. This results in a load magnitude of 30 kN/m^2 if b = 5 m with a uniform distributed load of 30 kN/m if a 1 m transverse strip is considered.

The variables "**a**" and "**c**" are defined in TMH 7.

"a": trailer length which varies in 5 m increments starting at 5 m.

"c": distance between trailers which varies from 0 m to 25 m in 5 m increments.

MAXIMUM SHEAR FORCE AND BENDING MOMENT

Maximum Shear Force (SF) and Bending Moment (BM) Due to Traffic Load

The individual maximum SF and BM were obtained by applying each load condition described in the bridge loading section to the 10 m and 22 m girders, together with the appropriate material and load factors. The maximum values per load condition and span are presented in Table 2.

It is important to note that the factored SFs and BMs in Table 2 are only due to the traffic loads acting on the girders. From Table 2, the maximum SF and BM for the 10 m spans were obtained as 238 kN and 495 kNm, while 436 kN and 2 396 kNm were obtained for the 22 m spans. These results varied by less than 2% from those determined by the consulting engineer. The SF and BM due to the self-weight must also be determined in order to design the steel girders.

Table 2. Summary of the traffic loading

Load cases	Data for 10 m span	Data for 22 m span
SF_{MAX} – NA conditions 1 & 2	186 kN	269 kN
BM_{MAX} – NA conditions 1 & 2	466 kNm	1 707 kNm
SF_{MAX} – NA conditions 3	165 kN	165 kN
BM_{MAX} – NA conditions 3	413 kNm	908 kNm
SF_{MAX} – NB loading	238 kN	367 kN
BM_{MAX} – NB loading	481 kNm	1 712 kNm
SF_{MAX} – NC loading	198 kN	436 kN
BM_{MAX} – NC loading	495 kNm	2 396 kNm

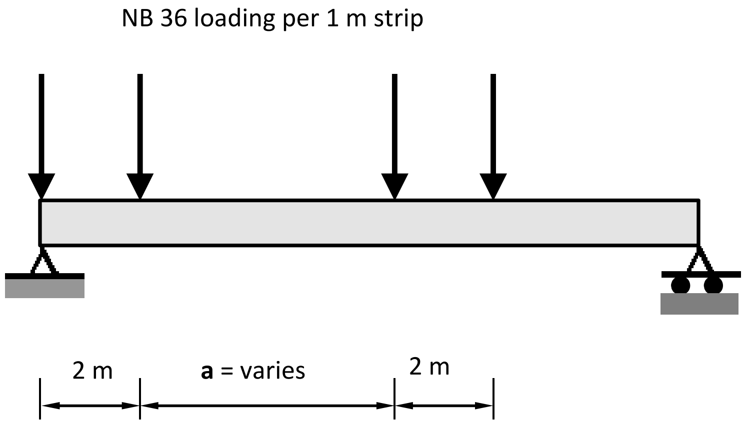


Figure (6): NB loading configuration

Maximum Shear Force and Bending Moment Due to Slab and Asphalt

The maximum SF and BM due to the self-weight of the 245 mm concrete slab and 50 mm of asphalt were also determined. Densities of 25.5 kN/m^{3}and 22.0 kN/m^3 were used for the concrete and asphalt, respectively, to obtain the uniform distributed load acting on the beam. Using the appropriate material factor specified in TMH 7, the maximum SF and BM for the 10 m span were obtained as 44 kN and 110 kNm, while 97 kN and 532 kNm were obtained for the 22 m beam.

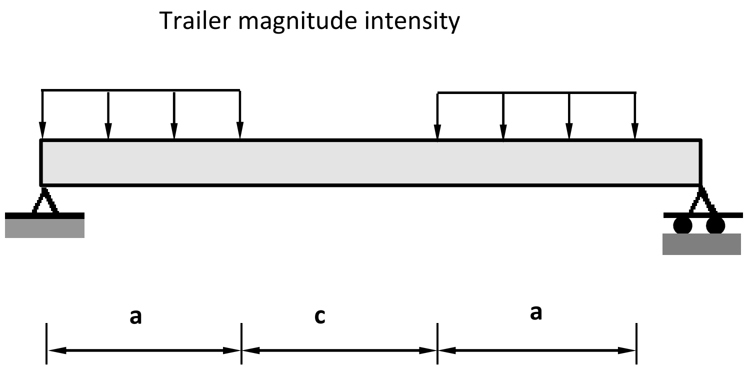


Figure (7): NA loading configuration

DESIGN OF THE STEEL GIRDERS

Initially, the sizes of the beam sections were approximated based on the maximum SF and BM due to the traffic load and the dead load of the bridge deck, excluding the self-weight of the beam. The combined maximum SF and BM of the traffic and dead loads for the 10 m beam are 282 kN and 605 kNm, while the 22 m beam yielded 530 kN and 2 928 kNm.

The steel girders were designed according to South African National Standard 10162-1: 2005. An iterative approach was employed to obtain the optimal steel section. For each case, once the approximate section was obtained, the moment of resistance was determined accurately, taking the self-weight of the steel section into account. The beams were designed to be laterally supported to allow for greater moment resistance capacity. This was achieved by creating a recess to the bottom surface of the concrete bridge deck to be at the same level of the top flange's bottom surface as shown in Figure 8.

To obtain the most economical design, the 10 m and 22 m beams were designed for a 1 m strip width, as well as for non-composite and composite action. For composite action, shear connectors were designed to resist the horizontal shear that forms between the concrete and the steel sections.

The shear resistance of each section was designed to be larger than the maximum applied shear. Where required, any single web section with a web-to-thickness ratio greater than 1:100 yield strength was supported with bearing stiffeners at the ends of the section. The bearing resistance of all plate sections was tested against web crippling and yielding.

It was necessary to design plate girders for the main spans, since the maximum SF and BM due to the combined loading were greater than the maximum moment of resistance of the largest commercially available I-section.

Summary of Steel Beams

Table 3 summarizes the most economical steel sections which satisfy both ultimate limit state and serviceability limit state requirements for non-composite and composite action. It should be noted that the design of the girders is controlled by the serviceability limit state requirements; i.e. deflection criteria. A span/deflection criterion of 300 was used in all cases.

Figure 8 shows an annotated cross-section through the composite girders with the bridge deck. The dimension annotations in Figure 8 can be obtained from the last 4 columns in Table 3.

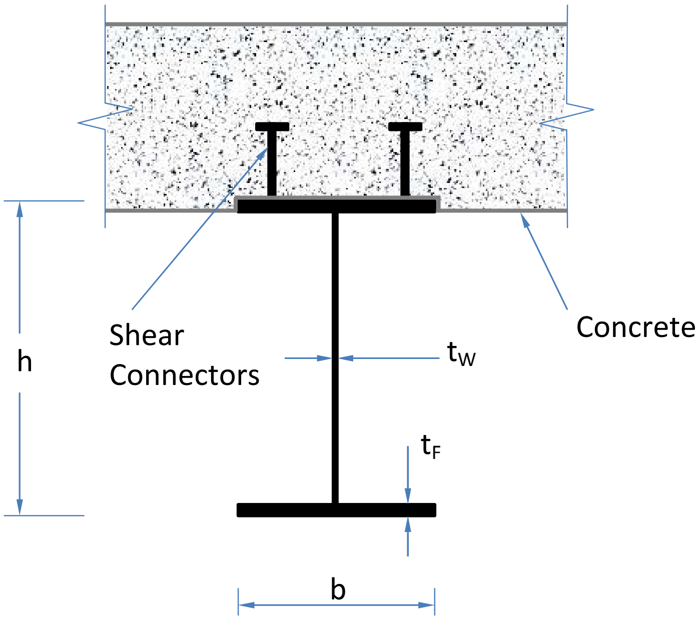


Figure (8): Cross-section through composite bridge girder

COSTING OF STEEL SECTIONS AND PLATE GIRDERS

Cost of the Steel Sections for the End Spans

Several steel suppliers within a 10 km radius from the site were contacted to provide a cost history over a 5-year period. Only one supplier could provide a cost history for off-the-shelf I-sections since 2007 at regular intervals during each year. Since the supplier has branches throughout South Africa and is a leading supplier of steel products, the data was used to conduct a cost analysis. The supplier's prices were also comparable to those obtained from other suppliers.

I-sections are rolled in 22 m lengths but are only available in 9 m and 13 m lengths. A 13 m length was required even though the end spans were only 10 m long. The cost of the girders was therefore based on the purchase of 13 m lengths.

Cost of the Plate Girders for the Central Spans

In a similar manner, quotations were sourced from the closest suppliers to Kuils River who manufacture plate girders. These suppliers were unable to provide accurate cost estimates of plate girders over a five-year period. Instead, only two suppliers provided quotations for the specific plate girders based on 2013 prices. They recommended a depreciation of between 7% and 8% to obtain cost estimates for the preceding years. The authors are aware that this would not provide an accurate comparison, as the escalation does not take the fluctuation of steel prices into account. However, it should provide some basis for comparison.

Cost of Protective Steel Treatment

Corrosion is the overriding cause of deterioration of steel, which reduces the amount of steel and thereby reduces the load-carrying capacity of the section. Corrosion occurs erratically on steel sections, which results in increased uncertainty with regard to the steel section's performance (Kayser and Nowak, 1989). Therefore, various corrosion treatments were investigated to protect the steel against deterioration over its intended design life of 50 years. These included painting of the steel sections using various commercial products and hot dip galvanizing. The disadvantage of painting is that, at best, the steel sections would have to be recoated at least four times over the intended design life. Before the section can be repainted, it must first be thoroughly cleaned. This, together with labour costs over the design life, makes this option less attractive.

Table 3. Summary of girders with geometric properties

Action Type	Span m	Section Designation	Shear Connectors	Bearing Stiffeners	Transverse Stiffeners	Total Section Height (h) mm	Flange Width (b) mm	Flange Thick-ness (t_F) mm	Web Thick-ness (t_W) mm
Non-Composite	L = 10	533 x 210 x 122 Off-the-shelf I-section	N/A	N/A	N/A	546	212	21.3	14.7
	L = 22	1230 x 400 Plate girder (8W, 25F)	N/A	110 x 12 mm	90 x 10 mm at 1.48 m spacing	1 230	400	25	8
Composite	L = 10	457 x 191 x 89 Off-the-shelf I-section	19 mm ϕ studs at 0.5 m spacing	19 mm ϕ studs at 0.5 m spacing	N/A	464	192	17.7	10.6
	L = 22	1040 x 400 Plate girder (8W, 30F)	19 mm ϕ studs at 1.0 m spacing starting 0.5 m from end	130 x 16 mm	60 x 6 mm at 2.2 m spacing	1 040	400	30	8

Galvanizing is the most effective method of protecting steel from corrosion. This process requires that steel sections are dipped into molten zinc at a temperature of approximately 460^{o}C. Zinc covers the steel to prevent corrosion. It also reacts with oxygen in the atmosphere to form zinc oxide, which further prevents corrosion (Ryan, 2009). The Hot Dip Galvanizers Association of Southern Africa (HDGASA) produces useful information for determining the life expectancy of hot dipped galvanized sections, which is based on coating thickness and environmental conditions. Based on the environmental conditions of the bridge in question and a 50-year lifespan, Wilmot (2013) suggested that a 150 μm zinc coating be used. Once the required zinc thickness is obtained, no maintenance is required on the installed steel sections, which makes this protective treatment more acceptable than painting. The current cost of hot dip galvanizing of steel sections is approximately $625 per ton. A depreciation of 10% was applied to obtain the galvanizing costs in the preceding years.

Cost of Shear Connectors

Shear connectors are required to achieve a composite action between the concrete and steel girders. Shear connectors of 19 mmϕ x 100 mm at 0.5 m spacing were required to achieve composite action. The maximum cost obtained from three suppliers was $2.90 per stud in 2010.

Transportation Cost

Suppliers could not provide a cost history for transporting steel sections per ton over the required period. The average current cost obtained for transporting steel per ton is $450.A depreciation of 10% was applied to obtain the transportation costs in the preceding years.

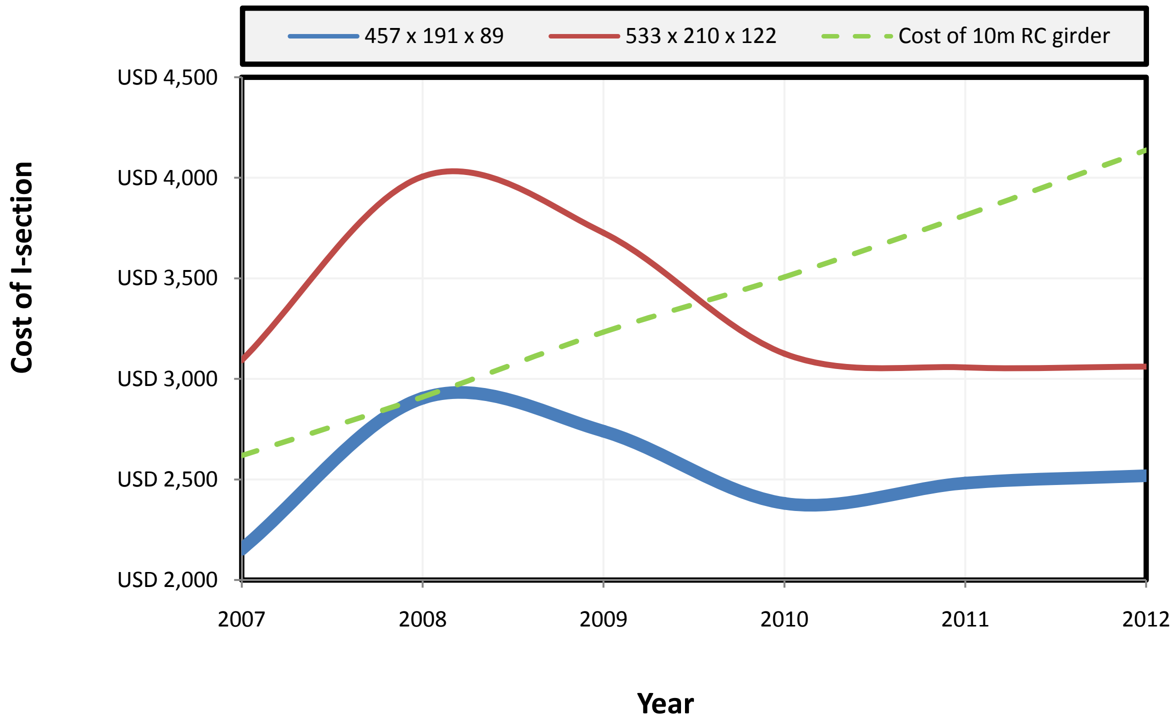


Figure (9): Cost of 10 m girders

Crane Hire Cost

Several crane hire companies were contacted to supply a quotation to hoist steel girders on the bridge piers. One supplier provided a quotation inclusive of the crane hire, VAT, insurance and site establishment. To obtain a cost estimate, the authors allowed 2 hours to place each beam in position. The current cost to hire a crane to hoist each beam was estimated at $190. A depreciation of 10% was applied to obtain the crane hire costs in the preceding years.

RESULTS AND DISCUSSION

10 m End Girders

The RC girders were manufactured 8 km off site by the contractor and therefore needed to be hoisted onto a low-bed trailer, transported to site and finally hoisted onto the bridge piers. The all-inclusive cost for the above was obtained from the contractor in 2009. The contractor suggested a 10% appreciation and depreciation to obtain a cost history for the RC girders.

Figure 9 presents the cost of the 10 m RC girders over a period of 5 years. Superimposed on Figure 9 is a cost history for off-the-shelf steel sections used for composite and non-composite construction. These costs are inclusive of the actual cost of the steel sections, galvanizing, transportation, crane hire and shear connectors where necessary.

Figure 9 confirms the significant price escalation of structural steel during 2008,which is also observed in Figure 3. The thick blue line and thin red line in Figure 9 represent the cost of using a beam for composite and non-composite construction, respectively. From Figure 9, it is observed that using steel girders in composite construction was more economical than using RC girders over the five-year period. An insignificant difference in the cost between composite construction and RC girders was observed at the beginning of 2008. However, apart from this time period, it was more economical to use composite construction due to the significant cost saving.

From Figure 9, it is also observed that RC girders were more economical than non-composite construction until mid-2009. Thereafter, RC girders became significantly more expensive than non-composite construction.

Table 4 shows the cost saving for composite and non-composite construction over RC girders for the 10 m end span, if a total of 20 beams were used. The shaded cells highlight when steel girder construction is more economical. From Table 4, it is clear that steel girders in short-span bridge construction are a viable option. It is also observed that the cost difference increases with time; i.e. RC girders became more expensive towards the end of the investigation.

22 m Central Span Girders

Figure 10 presents the estimated cost of steel plate girders over a five-year period. The authors are aware that the costs of the plate girders are approximate as they are based on 2013 prices. The preceding year estimates were obtained by applying an 8% reduction to the cost of the plate girders, as informed by a supplier.

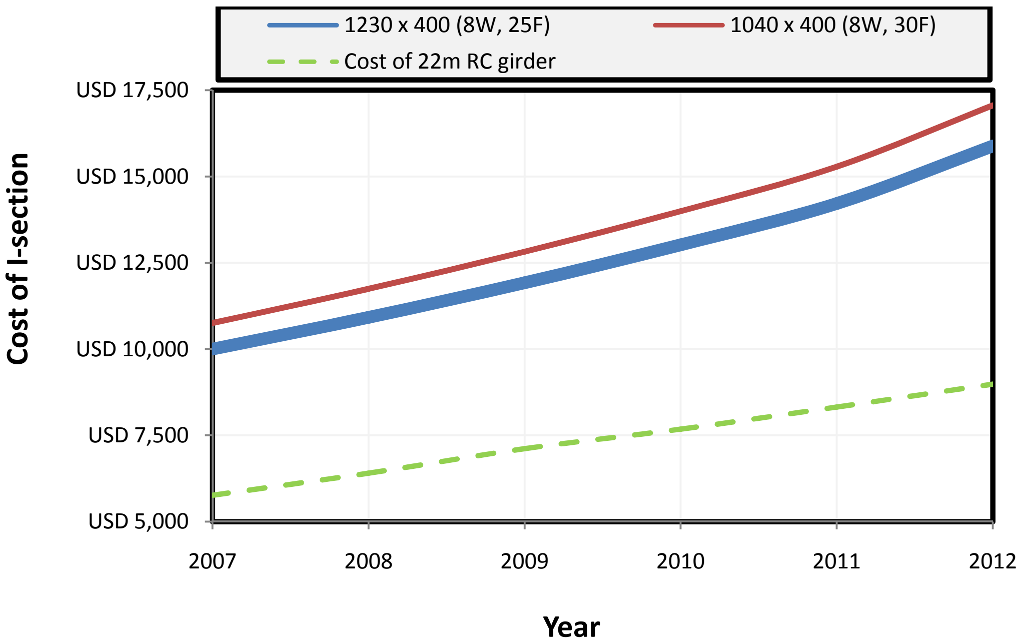


Figure (10): Cost of 22 m girders

Table 4. Total cost saving

Year	Total cost saving for composite construction	Total cost saving for non-composite construction
2007	$9 306	$9 461
2008	$114	$21 939
2009	$9 875	$9873
2010	$22 506	$7 622
2011	$26 650	$15136
2012	$32 376	$21 521

Based on Figure 10, it is clear that during the period considered the cost of manufacturing steel plate girders was significantly higher than that of post-tensioned RC girders. This is due to the cost of labour, welding, manufacture and wasted steel. Non-composite and composite construction are respectively on average 71% and 84% more expensive than post-tensioned RC girders. This undoubtedly shows that plate girders are not a viable option to use in bridge construction due to their uneconomical manufacturing costs.

CONCLUSIONS

The investigation reveals that for short-span bridge girders, of up to 10 m, it is imperative to perform a cost analysis to determine which construction material is most economical at the design stage. When the bridge over the Kuils River freeway was constructed in 2009, a cost saving of approximately $10 000 would have been achieved if composite construction had been used. However, RC girders were approximately $10 000 cheaper than non-composite construction at that time. This could justify the consulting engineer's choice to use RC girders. If the bridge was constructed in 2012, a cost saving of approximately $32500 and $21500 would be achieved if composite and non-composite construction were used for the 10 m spans. Based on this investigation, it can be concluded that the use of steel girders in short-span bridges is generally more economical than RC girders, especially toward the later part of this study.

The investigation also reveals that if the end span is 13 m long, a 533 x210x122 I-section could be used in bridge construction for the worst loading condition, provided composite construction is used together with cambering the I-section by 37 mm.

The investigation also shows that it is uneconomical to use plate girders in bridge construction due to their significantly higher manufacturing costs. This could, however, change if larger off-the-shelf I-sections were available in South Africa as is the case in other countries like the USA and England. In the USA, the largest commercially available off-the-shelf I-section is a W1100 x 499, which has an overall height of 1 118 mm and a flange width of 405mm. The W1100 x 499 has a second moment of inertia which is approximately 18 times greater compared to the largest available off-the-shelf I-section in South Africa. Also, the elastic section modulus is approximately 8 times larger compared with the largest available off-the-shelf I-section in South Africa. The investigation reveals that the W1100 x 499 has a moment of resistance greater than the ultimate moment required for the 22 m span girders.

The average cost of manufacturing the steel plate girders used in this analysis is approximately $2500 per ton compared with the average cost of the off-the-shelf I-sections of approximately $1550 per ton (2013 prices). Based on this, it can be deduced that the manufacturing cost of plate girders is approximately 67% more expensive compared to off-the-shelf I-sections. If off-the-shelf I-sections with the same dimensions as the plate girders used in this study were to be rolled in South Africa, this could result in an economically viable option for bridge construction. If larger steel sections were rolled in South Africa as in other countries, this would avoid having to manufacture plate girders and thus make steel girder construction in freeway bridges more economically viable.

The authors are aware that the investigation is not all inclusive, as it does not take different locations, different traffic loading for various classes of roads, maintenance cost of the RC girders,... etc. into account. However, the investigation aims at educating designers that RC girders are not always cheaper and also to elicit debate on the subject. The study also hopes to encourage steel manufacturers to explore this market need and roll larger commercially viable I-sections.

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