WWW.IJRPR.COM

International Journal of Research Publication and Reviews

Journal homepage: www.ijrpr.com ISSN 2582-7421

Comparison Between Design of Simply Supported and Integral Superstructure

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ABSTRACT

This study presents a comprehensive comparison between the design approaches of simply supported and integral superstructures in bridge engineering. Both design methodologies are critically analyzed, focusing on their respective advantages, limitations, and suitability for different project requirements. The analysis begins by examining the principles underlying simply supported superstructure design, including load analysis, material selection, structural analysis, reinforcement design, and construction considerations. Emphasis is placed on the flexibility provided by bearings and expansion joints, allowing for movement in response to external forces and environmental conditions. In contrast, integral superstructure design is explored, highlighting its unique features such as the absence of bearings and expansion joints. The study delves into integral connection design, abutment design, load analysis, material selection, structural analysis, and construction considerations. Special attention is given to the challenges associated with accommodating movement without traditional joints. Furthermore, the study evaluates the advantages and limitations of each design approach. Simply supported superstructures are praised for their established design principles, cost-effectiveness for shorter spans, and ease of construction. However, concerns regarding maintenance requirements and vulnerability to wear and corrosion at joints are noted. On the other hand, integral superstructures are lauded for their reduced maintenance needs, enhanced durability, and improved safety. Nevertheless, challenges related to higher construction costs, technical complexity, and limited movement accommodation are acknowledged. Ultimately, the decision between simply supported and integral superstructures depends on various factors such as span length, site conditions, budget constraints, and long-term maintenance considerations. This study aims to provide valuable insights for bridge engineers, aiding them in making informed decisions to ensure the safety, durability, and cost-effectiveness of bridge projects

Keywords: SIDL – Super Imposed Dead Load, Ap – Area of Pre-stress tendons, UTS – Ultimate tensile stress

1. INTRODUCTION

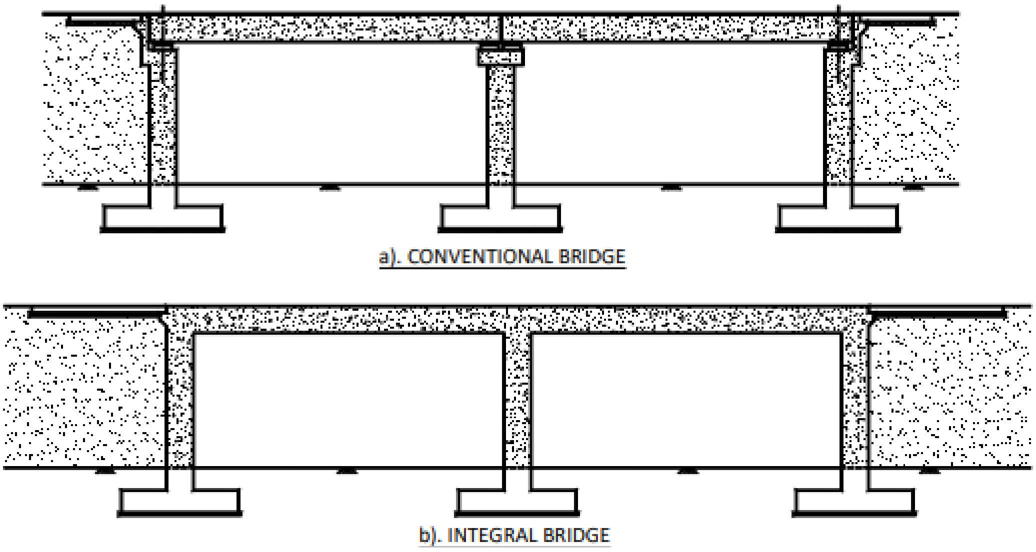
An Integral Bridge (IB) is a structure without bearings over the abutments and no expansion joints in the superstructure. Integral Bridge are characterized by a monolithic connection between the superstructure and the substructure, unlike traditional bridge construction where the superstructure is supported on bearings and transfers all the forces to the substructure and foundation through bearings. Expansion joints and bearings in traditional bridges allow movement and rotation of the bridge deck without transferring any force to the abutment/pier and foundation due to thermal/creep/shrinkage-induced movements. In the case of Integral Bridge, the deck carries the movement of the deck to the abutment as well as to the backfill soil behind the abutment. The approach slab between the bridge end and the pavements accommodates the necessary movements, leading to strong soil-structure interaction.

Other than bridges with complete integral solutions that don't have expansion joints or bearings, it's also possible to have a structural solution where only the expansion joints at the abutments are omitted, but the bearings are provided. In such cases, the back-wall portion of the substructure is directly connected with the superstructure and moves together with the superstructure, back-wall, and approach slab during thermal expansion and contraction, in relation to the backfill. These solutions, called "Semi-Integral Bridges" (SIB's), are often suitable, especially for rehabilitating bridges.

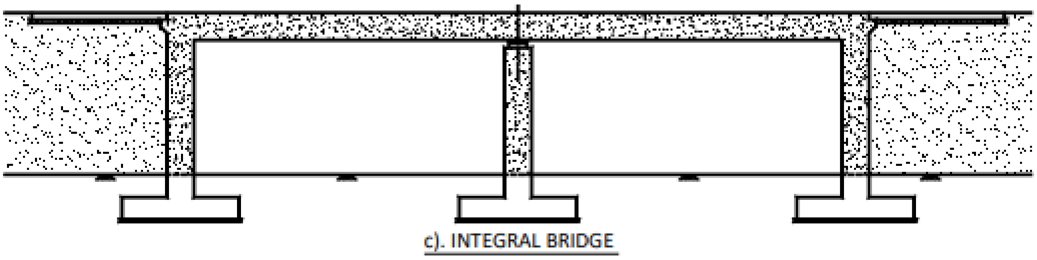
Another commonly used structural form is the Framed-Type Bridge (FTB), where the bridge deck is monolithic at intermediate pier locations but has bearings and expansion joints at the abutment locations. In this case, there is no interaction of the structure with the backfill soil. The design of FTBs is well covered in existing IRC codes and is therefore not covered in this guideline.

Fig. 1.1 illustrates the different types of bridges, categorized based on the connection of the deck at the ends.

There are four basic ways to make a bridge integral, depending on the abutment detail. These four forms can be referred to as bank seat abutments, framed abutments (fully integral bridges), embedded wall abutments, and flexible support abutments. Figure 1.2 shows typical details of these different types of integral bridges



(TYPE-1: Monolithic with Pier and Abutment).



(TYPE-2: Monolithic at Abutment and Bearing at Pier)

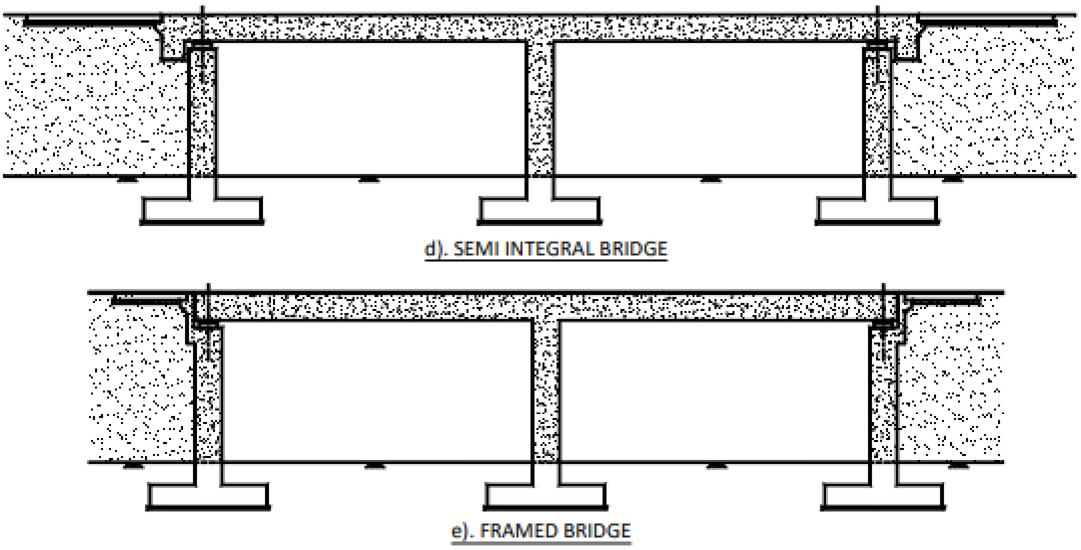


Fig. 1.1 Different Bridge Types Classified Based on Connection of Deck Ends

2. LITERATURE REVIEW

The construction of Integral Bridges in the United States of America began in the late 1930s and early 1940s. Ohio, South Dakota, and Oregon were the first states to routinely use continuous construction with integral abutments, and California followed suit in the 1950s. The shift towards integral bridges in Tennessee and other states began in the 1960s.

New Zealand has been building joint-less bridges since the 1930s, with standardized concrete bridge designs developed by the New Zealand Ministry of Works and Development (NZMWD) in the 1950s. During the 1970s, British researchers began studying integral bridges (IB). As of today, in the UK, bridges with a span length of less than 60 meters and a skew not exceeding 30 degrees are typically required to be continuous over intermediate piers and integral at abutments. The thermally induced cyclic movement at each abutment is limited to ±20 mm for IBs according to the British Advisory note.

In Japan, the first integral bridge was built in 1996. Integral bridge lengths in Japan are generally restricted to 30 meters. The Queensland Main Roads Department (QMRD) in Australia has been practicing integral bridge construction since 1975. China began building integral bridges in the 1990s.

The concept of integral bridges in Europe started in the 1960s. Since then, Europe has had a positive experience with them and there is a trend towards building more integral bridges in the region. Switzerland, for example, constructed many integral bridges on the national motorway network between 1960 and 1985. These bridges have been successful in terms of construction and maintenance. Currently, more than 40% of the bridges on the FEDRO (Federal Roads Office of Switzerland) network are integral or semi-integral structures. Researchers at EPFL (Ecole Polytechnique Federale de Lausanne) are working on building long span bridges with integral concepts, specifically bridges longer than 200m, by focusing on abutment and approach slab construction techniques.

2.1 Experimental Component Testing of Pile-Abutment Connection

In continuation of the research conducted by Greimann et al. (1984), Greimann et al. (1987) carried out pile field tests at one-tenth scale and full scale. These experiments involved applying lateral loads to the piles and measuring the resulting strains and displacements. The study confirmed and modified the guidelines for analytical modeling. Arsoy et al. (2002) conducted tests on three different types of piles: an HP10x42 pile, a 14-inch concrete-filled tube (CFT14), and a 12-inch prestressed concrete pile. The purpose of the tests was to assess the expected lifespan of integral bridges under typical working conditions by subjecting the piles to lateral load cycles caused by realistic displacements resulting from annual temperature differentials over a 75-year period. The study recommended the use of HP piles for weak axis bending to minimize stresses on the abutment. It found that the H-pile showed no degradation over the simulated 75-year period. However, the study did not recommend the use of prestressed concrete piles in integral abutment bridges. Furthermore, the test setup was unable to accommodate the CFT 14. Chovichien (2004) conducted full-scale tests on various pile sections commonly employed in integral abutment bridges. The pile sections tested included six HP sections and three concrete-filled tubes (CFT). They were subjected to testing for weak axis, strong axis, and 45° axis bending, assessing their lateral deformation and strength capacity. Through additional analytical modeling, Chovichien (2004) established maximum lateral deformation guidelines for typical pile sections and different soil conditions utilized in integral abutment bridges. The study concluded that the maximum lateral deformation capacity for typical pile sections in integral abutment bridges is 2 inches, and recommended the use of piles in weak axis bending. Talbott (2008) expanded on Chovichien's work from 2004 by conducting tests on additional HP pile sections. The additional tests revealed that two damage limits could be established for HP sections: the zero damage limit and the acceptable damage limit. The zero damage limit refers to the maximum allowable lateral deformation that results in no damage to the pile, which aligns with the 2-inch limit defined by Chovichien (2004). The acceptable damage limit pertains to deformation that causes less than a 5% loss of load carrying capacity, corresponding to a 4-inch allowable deformation for HP sections commonly used in integral abutment bridges.

2.2 Effects of Abutment Soil

Duncan and Mokwa (2000) conducted a study on current models used to predict passive earth pressure and their suitability for abutments and laterally loaded pile caps. Their research identified the log-spiral method as the most effective technique for predicting lateral earth pressure. Additionally, Duncan and Mokwa developed a method for modeling the load path of passive earth pressure, enabling designers to determine pressures between static and full passive based on displacement into the fill. Rollins and Cole (2006) conducted a study on the cyclic lateral load behavior of pile caps. In their research, they tested seven full-scale pile caps, four of which had backfill at varying compacted levels. Their findings offer valuable insights into the modeling of backfill material and are relevant to integral abutment bridges.

2.3 Full-Scale Modelling of Integral Abutment Structures

In 1993, Girton and others conducted a two-year field investigation of two integral abutment structures: the Boone River Bridge, a 324.5 ft prestressed girder bridge with a 45° skew, and the Maple River Bridge, a 320 ft steel-girder bridge with a 30-degree skew. The study focused on monitoring longitudinal displacement of the abutments, deck temperatures, and pile strains. However, it didn't include direct measurements of transverse movements. They developed a longitudinal analytical model (simple frame) using equivalent column methods by Abendroth (1989) and compared the results with field measurements. Additionally, they coupled a transverse model (simple frame) with strain measurements on selected piles to predict transverse movements. The research concluded that Abendroth's equivalent column method effectively represents the longitudinal behavior of the pile. It also advised designers to carefully consider lateral movement in skewed structures but didn't provide specific recommendations for determining the magnitude of transverse movement.

In 2000, Lawver et al. conducted a field monitoring program on a 216.5 ft prestressed girder bridge without skew for approximately two and a half years. The structure was extensively instrumented to monitor temperature, lateral displacement of the abutment, pile strains, earth pressure, and pier movement. A live load test was also conducted as part of the investigation. The study made numerous observations about the behavior of integral abutment bridges. Particularly significant was the observation that the abutment underwent a net inward movement during each annual cycle.

Brena et al. (2007) carried out a three-year monitoring program on a 270-foot steel plate girder bridge with zero skew. The structure was extensively instrumented with pile strain gauges, inclinometers, and earth pressure cells. The investigation led to various conclusions regarding the behavior of integral abutment bridges. It was found that abutments undergo rigid body motion involving both rotation and translation. This behavior leads to lower moments in piles, which are usually designed to be fixed against rotation. Additionally, it was observed that the bridge experienced 60% of the displacements predicted by unrestrained thermal shrinkage. An investigation conducted by Chovichien (2004) involved three integral abutment structures.

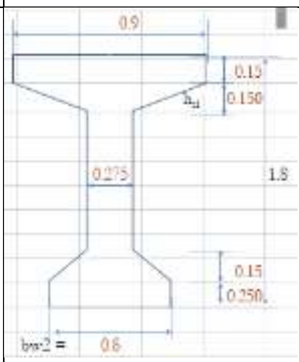
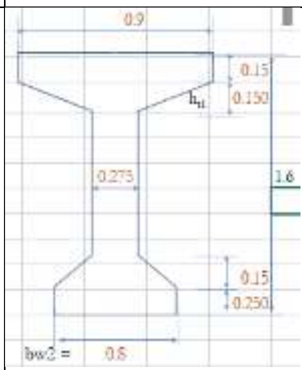
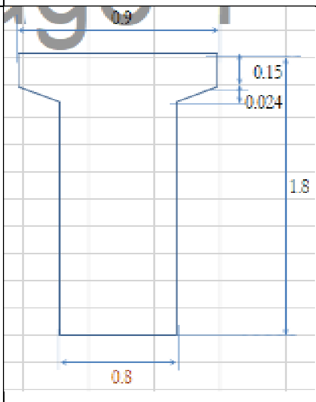
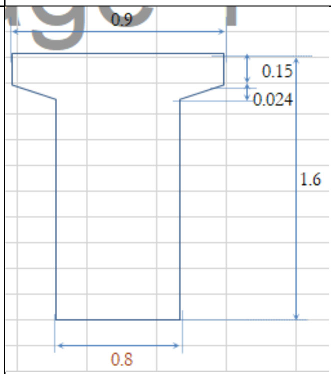
The monitoring program covered a 152 ft steel-girder bridge with a 25° skew, a 367 ft prestressed girder bridge with an 8° skew, and a 990 ft prestressed girder bridge with a 13° skew. The monitoringq program for these structures began in Summer 2000, Summer 2003, and Spring 2000, respectively.

3. METHODOLOGY

A simply supported superstructure and integral bridge have been modelled in Staad Pro. Connect Edition. The span length, width and numbers of girders are considered same in both of cases. The following methodology is adopted during analysis:

3.1 Geometry selection

Table 3.1Following are geometry details of the simply supported and integral span

S. No.	Description	Simply Supported	Integral Superstructure
1.	Span	30.0m(Exp to Exp Gap)	30.0(CL to CL of pier shaft)
2.	Module	Single span module	Three span modules
3.	Type of superstr	I- Girder	I- Girder
4.	Substructure	Not dependent	Single row pier dia 1.2
5.	Foundation	Not dependent	Single row pile dia 1.2
2.	Deck Width	10.5m	10.5m
3.	Nos. of Girder	4	4
4.	Spacing b/w gir	3.0m	3.0m
5.	Deck Cantilever	0.75m	0.75m
6.	Girder Section-Mid		
7.	Girder Section-Support		
8.	Deck Thick	0.22m	0.22m

3.2 Construction Sequence

Table 3.2 Following are Construction Sequenceof the Simply supported and Integral Span

S.No.	Item	Day No.
1	Completion of the casting of PSC Girder	0
2.	Prestressing of tendon for stage-1 Cable	14 Days after casting of girder or when girder concrete attained a strengthof45Mpawhicheveris later
3.	Prestressing of tendon forstage-2 cable	28 Days after casting of girder or when girder concrete attained a strengthof50Mpawhicheveris later
4.	Casting of deck slab & diaphragm	30Daysaftercastingof girder
5	Application of crash barrier, wearing Coat etc.	45daysafter casting of girder

3.3 Grillage Idealization

The longitudinal grid lines for I-beam decks is to make them coincide with the center lines of physical girders and these longitudinal members are given the properties of the girders plus associated portions of the slab, which they represent.

The transverse grid lines i.e. deck slab is conceptually broken into a number of transverse strips and each strip is replaced by a grid line. The spacing of transverse grid lines is somewhat arbitrary but about 1/8th of effective span is generally convenient. As a guideline, it is recommended that the ratio of spacing of transverse and longitudinal grid lines be kept between 1 and 2 and the total number of lines be odd. (Reference – Grillage Analogy in Bridge Deck by Surana and Agrawal).

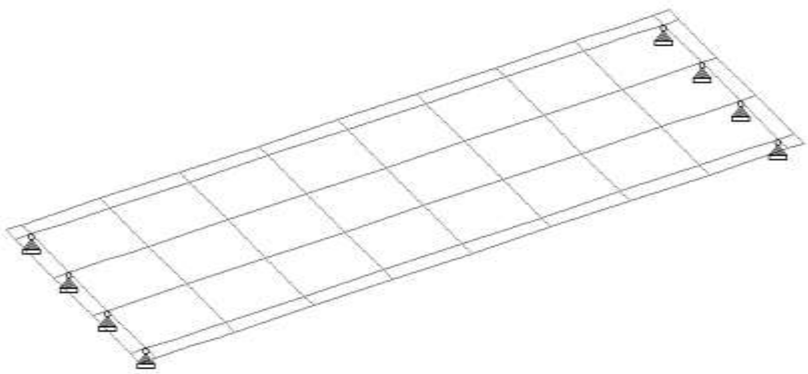


Figure 3.1: Simply supported superstructure

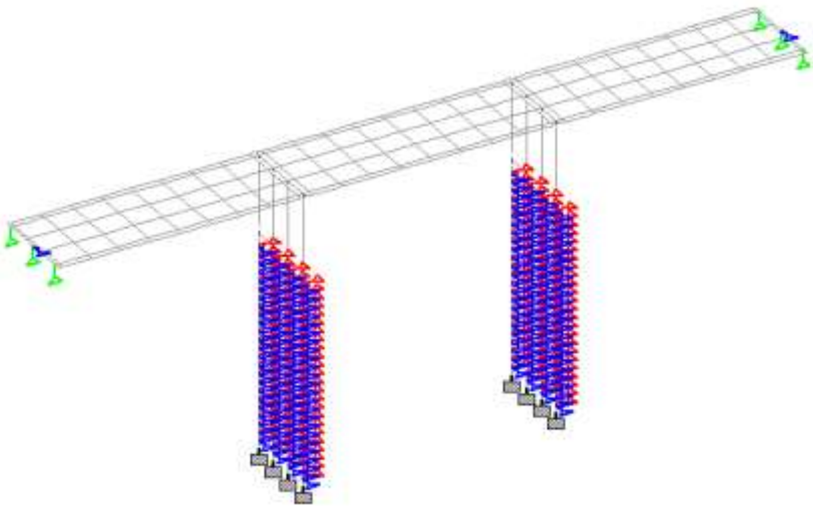


Figure 3-2: Integral superstructure

3.4 Section Properties

Table 3.3 Details of Section Properties

	Description	Simply Supported	Integral Superstructure
1	Composite Proper- Support	A = 1.925m^2 IYY=0.280m^4 IZZ= 0.757m^4	A = 1.777m^2 IYY=0.2628m^4 IZZ= 0.568m^4
2	Composite Proper- Mid	A=1.275m^2 IYY=0.2026m^4 IZZ = 0.630m^4	A=1.232m^2 IYY=0.2167m^4 IZZ = 0.484m^4

Table 3.4: Type of Support Condition

	Description	Simply Supported	IntegralSuperstructure
1	Support condition	Pin Support defined	Soil subgrade reaction defined for foundation of integral pier. At simply supported location, longitudinal direction – Outer bearing free and stiffness of inner bearinghasbeenassigned basedonpier stiffness.

Table 3.5 Applicable Load Cases

S. No.	Load	Simply supported case	Integral case	Remarks
1.	DL of Precast Girder	√	√	Load intensity = Cross section area * density of concrete (2.5 t/m^3)
2.	Green weight of deck	√	√	Load intensity = Cross section area of deck * green density of concrete (2.6 t/m^3)
3.	Redistribution of load due to composite action	√	√	Load intensity = Cross section area of deck * green density of concrete (2.6 t/m^3)
4.	Prestress force due to cable	√	√	All prestressing strands shall have 7 ply uncoated stress relieved low relexation high tensile strands of 12.7mm dia conforming to class 2 of IS 14268 – 1995. Cable type 19-T-13/S system detail: Ultimate Tensile stress of one strand = 1861 N/mm^2 Nominal Area of strand =

S. No.	Load	Simply supported case	Integral case	Remarks
				$98.7\ mm^2$ Ultimate breaking strength of one strand = 183.7 KN Total Nos. of strand = 19 UTS of cable = 356.1 tons Maximum allowable force in the cables before anchoring = 0.765 x UTS =272 tons
5.	Immediate loss (friction, slip & elastic shortening) in prestress force	√	√	Cable profile – Linear at mid, parabola and straight at ends. According friction has been calculated. Slip is considered 6mm.
5.	Creep, shrinkage & relaxation for time I[st] to II[nd] stage stressing	√	√	Calculated based on properties of only girder.
6.	SIDL	√	√	Crash barrier on both side of the carriageway having load intensity of 0.8 t/m.
7.	SIDL -WC	√	√	Applied load of intensity 0.2 t/m2 considering future overlaying.
8.	Redistribution of loads applied before integral	X	√	Redistribution of loads are dependent on creep factor. The load is calculated as creep factor*(pre – post loading).
9.	Uniform temp rise/fall	X	√	Applied based on codel provisions of IRC-6
10	Temp gradient	√	√	Applied based on codel provisions of IRC-6
11.	Differential shrinkage	√	√	Differential shrinkage will occur due to variations in the time of casting of the girder and deck.
12.	Live load	√	√	IRC loading – 70R wheeled loading, Class A loading & SPV loading.
13.	Creep, shrinkage & relaxation loss in prestress force.	√	√	Calculated based on specifications of IRC: 112-2011.

Table:3.6 Coefficient of friction (μ) & Wobble Effect (k) of Post Tensioned tendons and External Unbounded Tendons (Ref. IRC: 112-2020)

Type of High Tensile Steel	Type of Duct Or Sheath	Values recommended to be used in design	
		Kpermeter	**μ**
Wire cables	Bright, metal steel	0.0091	0.25
	Galvanised steel	0.0046	0.20
	Lead coated steel	0.0046	0.18
	Unlined duct in concrete	0.0046	0.45
Uncoated Stress Relieved Strands	Bright Metal Steel	0.0046	0.25
	Galvanised steel	0.0030	0.20
	Leadcoated	0.0030	0.18
	Unlined duct in concrete	0.0046	0.50
	Corrugated HDPE	0.0020	0.17

Relaxation loss occurs due to gradual decrease in the pre-stress force in the tendon over time dueto the relaxation of the material. For long-term losses, the design value of relaxation can be approximated by taking three times the 1000-hour value at an initial stress of 70% UTS.

Table:3.7 Relaxation for other Values of Initial Stress (Ref.IRC:112-2020) (Expressed as percent of initial stress tested at 1000 hours at 200C ± 20C)

Initial Stress	Relaxation loss for Normal relaxation steel (%)	Relaxation loss for low relaxation steel (%)
≤0.5 fp	0	0
0.6 fp	2.5	1.25
0.7 fp	5.0	2.5
0.8 fp	9.0	4.5

Table 3.8 Relaxation Loss Upto 1000 Hours (Ref.IRC:112-2020)

Time In Hours		1	5	20	100	200	500	1000
%loss of 1000hrs. loss	Normal Relaxation Steel	34	44	55	70	78	90	100
	Low Relaxation Steel	37	47	57	72	79	90	100

4. CONCLUSIONS

Table 4.1 Summary of bending moments areas

S.No.	Load Case	Simply supported case(t-m)	Integral case(t-m)
1.	DL of pre-cast girder	217.77	194.6
2.	DL of deck girder	175.49	175.49
3.	SIDLCB	27.58	17.4
4.	Surfacing	45.92	13.3
5.	Without SPV	295.67	166.8
6.	With SPV	391.07	231.2

As per above table, bending moments due to load applied in integral bridge are significantly lower with respect to bending moments developed in simply supported case

Table 4.2 Summary of cables areas

S. No.	Stage	Number of stands				
		Cable-1	Cable-2	Cable-3	Cable-4	Total
1.	Simply supported	19	19	19	11	68
2.	Integral span	19	19	18	-	56

The number of strands required for an integral bridge is lower than for a simply supported structure

Table: 4.3 The quantities of concrete and HT steel are shown in below table for both cases;

Case	Concrete Quantity(Cum)	HT Steel Quantity(tons)
Simply supported girder	27.30	15.62
Integral girder	25.10	12.86

The integral bridge requires lower quantities of concrete and HT steel compared to the simply supported structure

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