Article

Dynamic Performance of a New-Type PSC I-girder for Railway Bridge Application

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Abstract: This study intends to verify analytically and experimentally the performance of a new type of prestressed concrete (PSC) I-girder for its application as railway bridge. Since the girder-type railway bridge develops relatively low torsional rigidity, there is risk for the dynamic responses to amplify due to the superposition of the torsional mode and flexural mode. The superposition of the torsional and flexural modes as well as the dynamic stability of the railway bridge were examined through dynamic analysis. Three-dimensional modelling was built to be suitable for carrying out moving load analysis. Four different span lengths of 30, 35, 40 and 45 m adopted considering the most applied span length currently and future lengthening of the span length. Moreover, a full-scale girder specimen with span length of 35 m was fabricated and subjected to dynamic loading. The measured dynamic responses were then compared to the analytic values. Finally, the ultimate bearing capacity of the specimen was verified by static loading test.

Keywords: PSC I-girder; railway bridge; medium span length; moving load analysis; dynamic characteristics

1. Introduction

Korea achieved remarkable growth of its transportation infrastructure within a very short period. The Korean government annually announces the current status of roads and railroad facilities [1,2]. Figure 1 is reconstructed using current status data for roads [1] and railroad facilities [2]. Figure 1a plots the evolution of the inventory of the road and railway bridges during the decade running from 2008 to 2018. As of 2018, the extension of the roadway and railway reaches 114,788 km in which 40,435 bridges occupy 4198 km. Compared to 2008, 9129 new bridges were erected over 1238 km representing a 42% increase of the extension and a 29% increase of the bridge stock. In details, there are today 34,297 roadway bridges and 3138 railway bridges spanning over 3451 km and 747 km, respectively (Figure 1b).

Figure 1. Evolution of transportation infrastructure in Korea: (a) total length and bridge inventory; (b) proportions of roadway and railway bridges.

Table 1 classifies the 34,297 roadway bridges according to the type of their structure and superstructure [1]. The most preferred superstructure types are the reinforced concrete (RC) slab bridge followed successively by the rahmen (rigid-frame) bridge, the prestressed concrete (PSC) I-girder bridge and the steel box girder bridge with rates of occupancy of 24.1%, 23.6%, 22.6% and 13.7%, respectively. These four types of superstructure account for 84% of the bridge stock. According to Jung et al. [3], most of the bridge superstructures applied the RC slab and the RC T-girder in the 1950s, and the PSC I-girder became the preponderant type of superstructure after a peak in the 1960s owing to its low cost and shortened construction period. Figure 2a shows the domestic bridge market share of the PSC I-girder since 1970 and reveals that this share increased by an average of 36.2% between 2014 and 2018. Figure 2b presents the change in the distribution of the span length for the PSC I-girder since its first introduction in the early 1960s. It can be observed that the PSC I-girder is currently preferred for medium span lengths between 30 m and 40 m. Apart from special long-span bridges, the application of the PSC I-girder as superstructure is now common in Korea for its aesthetics and economy for usual span lengths [4,5].

Table 1. Distribution of bridges by superstructure type in Korea.

Bridge Type	Number of Bridges	Total Span (km)	Superstructure Type	Number of Bridges	Total Span (km)
Suspension	9	21.3	RC slab	8271	348.1
Cable-stayed	69	74.3	RC hollow slab	238	22.1
Extradosed	22	14.1	RC box girder	211	9.4
Truss	17	8.9	RC T-girder	751	36.0
Internal	8092	173.6	PSC slab	491	41.1
Arch	226	40.7	PSC hollow slab	40	7.8
Steel plate girder	307	61.2	PSC box girder	625	336.8
Steel box girder	4705	1051.7	PSC I-girder	7768	1009.3
Steel I-girder	201	23.1	Preflex girder	1362	88.4
Others	892	83.6	Total	34,297	3451.5

For the roadway bridges, the stiffness of the girder is a primary factor determining the structural stability. Besides, securing the dynamic stability under large live loads and repeated loading is the primary factor in the case of railway bridges. Accordingly, studies have been conducted recently on the

development of dedicated monitoring systems as well as on the evaluation of the dynamic behavior of high-speed railway bridges [6–8].

Figure 2. Yearly evolution of prestressed concrete (PSC) I-girder bridge inventory in Korea: (a) market share; (b) distribution per span length.

The present study intends to evaluate the structural behavior of a new type of PSC I-girder for its application to railway bridges. Since the I-girder exhibits relatively low torsional rigidity, it may produce large responses even under the natural frequencies for the same flexural mode because of its light weight, which necessitates precise evaluation of the dynamic stability [9]. The considered span length is decided based upon the analysis of the preferred span length in the design stage, and both analytical and experimental evaluations are carried out for the application of the new type of PSC I-girder in the railway bridge.

2. Dynamic Performance Evaluation of New Type of PSC I-girder Bridge

2.1. Structure Considered in Analysis

The new type of PSC I-girder considered in this study applies the half-slab to take full advantage of its characteristics in term of construction stability. The so-called half-slab girder of which upper flange plays the role of slab presents a reduced height thanks to the efficient arrangement of the prestressing tendon through the dualization of the anchors of the tendon by disposing intermediate anchors in addition to the end anchors as shown in Figure 3. Four different span lengths (30, 35, 40, 45 m) of the girder were adopted considering recent trends in the span lengths for the PSC I-girder. Figure 3 depicts the longitudinal plan of the new type of PSC I-girder. The sectional dimensions were determined by applying the Korean standard train load KRL-2012 [10] for passenger car mixed line in the design. Figure 4 shows the section of PSC I-girder designed to sustain two railway lanes. Four girders are applied for the span lengths of 30 m and 35 m, and five girders for the span lengths of 40 m and 45 m. The height of the girder is set to 2.5 m for the span lengths of 30 m and 35 m, 2.7 m for the span length of 40 m, and 3.2 m for the span length of 45 m. The thickness of the slab is 280 mm including the 140 mm-thick half-slab. Concrete track is adopted in the model. Table 2 arranges the material properties of the considered bridge. Concrete with compressive strength of 50 MPa is adopted for the girder and concrete with compressive strength of 30 MPa is adopted for the slab.

Figure 3. Longitudinal plan of railway bridge applying the new type of PSC I-girder.

Figure 4. Front view of railway bridge section: (a) application of 4 girders for span lengths of 30 m and 35 m; (b) application of 5 girders for span lengths of 40 m and 45 m.

Table 2. Material properties of concrete.

Compressive Strength (MPa)		Young's Modulus (MPa)		Density (kN/m ³)
Girder	Slab	Girder	Slab	
50	30	32.902	28.577	24.5

2.2. Performance Evaluation by Moving Load Analysis

2.2.1. Performance Evaluation Criteria

Compared to the road bridge, the railway bridge experiences larger live loads and may suffer dynamic problems like resonance due to the cyclic loading generated by the train wheels traveling at high speed. The railway bridge must thus satisfy thorough vibration limits as well as vertical deflection limits. Accordingly, the railway design code (KDS 24 10 10) requires to perform moving load analysis for the train load per range of traveling speed and to perform the evaluation of vertical acceleration, vertical deflection and twist of the bridge span in order to secure the dynamic stability of the railway bridge subjected to traveling trains. Table 3 details the limits for each check item specified in KDS 24 10 10 [11].

Table 3. Railway bridge specifications for traveling safety and comfort (KSD 24 10 10) [11].

Item	Application		Limit
Vertical acceleration	Gravel ballast track		0.35 g
	Concrete ballast track		0.5 g
Twist	Standard train load	$V \leq 200$ km/h	3.0 mm/3 m
		$V > 200$ km/h	1.5 mm/3 m
		Dynamic analysis with real train	1.2 mm/3 m
Vertical deflection	Span length	30 m	$L/1600$ ($270 < V \leq 350$)
			$L/1500$ ($200 < V \leq 270$)
			$L/1300$ ($V \leq 200$)
		35 m	$L/1750$ ($270 < V \leq 350$)
			$L/1600$ ($200 < V \leq 270$)
			$L/1500$ ($V \leq 200$)
		40 m	$L/1900$ ($270 < V \leq 350$)
			$L/1700$ ($200 < V \leq 270$)
			$L/1500$ ($V \leq 200$)
		45 m	$L/2100$ ($270 < V \leq 350$)
			$L/1900$ ($200 < V \leq 270$)
			$L/1400$ ($V \leq 200$)

The five different train loads listed in Table 4 were considered in the moving load analysis. These train loads correspond to those currently in operation or under plan. Here, the Saemaeul train and the freight train are conventional trains operating in the previous railway network and KTX (Korea Train eXpress) stands for the high-speed train. HEMU (Highspeed Electric Multiple Unit) and EMU (Electric Multiple Unit) are the high-speed trains of the next generation. Table 4 provides also the composition of the convoy, the size of the axle load and the effective beating interval for each of the considered trains. Figure 5 shows the KTX train, one of the five train models. Moving load analysis was performed at intervals of time until the train started from the point of the bridge and completely passed the bridge. At this time, the element was set to hit three or more times to ensure numerical stability.

Table 4. Characteristics of trains in operation or under plan in Korea.

Characteristics	Train in Operation			Train under Plan	
	KTX	Saemaeul	Freight	HEMU	EMU
Composition	20 coaches	8 coaches	20 coaches	12 coaches	8 coaches
Axle load (kN)	170~190	107.91~215.82	196.2	130	156.8
Effective beating interval (m)	18.7	23.5	13.95	24.3	21.4
Max. speed for analysis (km/h)	420	300	220	430	330

Figure 5. KTX Train.

The occurrence of resonance or resonance suppression in the railway bridge depends on the relationship existing between the effective beating interval and the span length [12–14]. Resonance occurs when the natural frequency of the bridge coincides with the frequency of the force vibration generated by the periodic beating of the regularly spaced axles of the train. The critical speed (V_{cr}) of the train causing resonance can be calculated by Equation (1) [11], where V_{scr} is the sub-critical speed; w_1 is the natural frequency of the first flexural mode of the bridge; and, S_{eff} is the effective beating interval of the train load as given in Table 4. Moreover, the sub-critical speed is obtained as an integer ratio of the critical speed as expressed in Equation (2). Besides, resonance suppression is conceptually opposite to resonance that deals with the suppression of the free vibration effect of the bridge caused by the traveling train. In other words, resonance suppression is a phenomenon that occurs when the dynamic response of the bridge is determined only by the load currently positioned in the bridge [14]. Equation (3) gives the span length $L(m)$ of the bridge for which resonance suppression occurs.

$$V_{cr} = w_1 \times S_{eff}, \quad (1)$$

$$V_{scr} = \frac{w_1}{n} \times S_{eff}, \quad n = 1, 2, 3, \dots \quad (2)$$

$$L(m) = S_{eff} \times (i + 0.5), \quad i = 1, 2, 3, \dots \quad (3)$$

The fact that the resonance suppression span length is determined with respect to the effective beating interval of the train indicates the existence of a span length for which the response at resonance is suppressed according to the train type. Figure 6 plots the resonance suppression span length provided by Equation (3) for each of the five considered trains.

Figure 6. Resonance suppression span length per type of train.

Accounting that the PSC I-girder is mostly applied to span lengths between 30 m and 40 m (Figure 2), the span length of 35 m appears to be the most favorable in term of resonance suppression. Consequently, this span length of 35 m is expected to develop small response under the traveling trains. Due to the discrepancy with the resonance suppression span length for KTX, evaluating the behavior of the bridge by moving load analysis will bear utmost importance. Besides, the span length of 30 m is the closest to the resonance suppression span length for KTX. In such a case, it will be necessary to conduct precise moving load analysis for HEMU and EMU. In addition, the satisfaction of the design criteria including the acceleration, displacement and twist for each of the train loads shall be checked by dynamic analysis.

2.2.2. Analysis Method

The dynamic moving load analysis is carried out using the in-house programs HFC and hfAnalyzer developed at Korea Institute of Civil Engineering and Building Technology (KICT). HFC is a kind of library designed by object-oriented programming that allows the user to implant his/her own material model and analysis method. The hfAnalyzer was developed using HFC and its applicability has been validated by applying various modeling and analysis methods on diversified railway bridges [9]. The model adopts 2-node beam elements for the girder and cross beams to reflect the features of the girder bridge and 4-node shell elements for the deck. The concrete ballast track composed of PCL (protection concrete layer) and TCL (track concrete layer) is also reflected in the model to achieve accurate analysis. Specifically, the PCL is modeled as a distributed mass load and TCL is modeled by solid elements. The rails are modeled by 2-node beam elements to consider the entrance and exit of the traveling train on the bridge. For the bridge bearings, beam-type restraints are applied with deck considering the gap with the girders. Figure 7 depicts the analytical FE model for the girder bridge with span length of 35 m. The damping ratio in the moving load analysis has the most sensitive effect on the amplitude of the response of the bridge at resonance. Therefore, this study applies a damping ratio of 1% as suggested in the Railway Design Code (KDS 24 10 10) [10,11].

The moving load analysis was carried out for the train since its access on the structure to its complete crossing for traveling speeds increased by step of 10 km/h within the maximum speed range and including the critical and sub-critical speeds listed in Table 4. The train load was applied on one line to consider the effect of eccentric loading, and the amplitudes of the vertical deflection, vertical acceleration of the deck as well the twist of the track were checked.

Figure 7. Model of PSC I-girder railway bridge with two traffic lines for moving load analysis.

2.3. Dynamic Performance Evaluation of New PSC I-girder Bridge

2.3.1. Natural Frequencies

Since the I-girder develops lower torsional rigidity than the box girder, its torsional mode will occur at proximity of the flexural mode in the lower order mode range, which presents the risk for these two modes to superpose and amplify the responses [9]. Table 5 lists the natural frequencies of the PSC I-girder bridge with the corresponding mode according to the type of train with their critical speeds per considered span length of the bridge. The first flexural mode appears to be the first mode for the span lengths of 35 m and 40 m whereas the first mode for the span lengths of 30 m and 45 m is the torsional mode. In addition, when the first mode is the flexural mode, the second mode is the torsional mode and happens closely. Figure 8; Figure 9 shows the first and second mode shapes for the span lengths of 30 m and 35 m. The first flexural mode related to the critical speed ranges between 3.39 Hz and 4.5 Hz.

Table 5. Analytically obtained natural frequencies and critical speeds per train type.

Span Length	Mode	Natural Freq. (Hz)	Critical Speeds (km/h)				
			KTX	HEMU	EMU	Freight	Saemaeul
30 m	1(Torsion)	4.14	278.81	362.31	319.07	207.99	350.38
	2(Flexure)	4.50	302.73	393.39	346.44	225.83	380.44
	3(Torsion)	7.16	482.26	626.68	551.89	359.76	606.05
	4(Torsion)	12.74	857.66	1114.50	981.49	639.80	1077.80
35 m	1(Flexure)	4.27	288.02	374.27	329.60	214.86	361.95
	2(Torsion)	4.67	314.87	409.16	360.33	234.89	395.69
	3(Torsion)	8.21	552.79	718.33	632.60	412.38	694.68
	4(Torsion)	13.14	885.10	1150.16	1012.90	660.28	1112.30
40 m	1(Flexure)	3.56	239.92	311.77	274.56	178.98	301.51
	2(Torsion)	3.76	253.23	329.06	289.79	188.91	318.23
	3(Torsion)	7.70	518.63	673.95	593.52	386.89	651.76
	4(Torsion)	11.35	763.81	992.55	874.10	569.80	959.87
45 m	1(Torsion)	3.29	221.51	287.84	253.49	165.24	278.37
	2(Flexure)	3.39	228.47	296.89	261.46	170.44	287.11
	3(Torsion)	7.66	515.39	669.73	589.80	384.47	647.68
	4(Torsion)	10.12	681.48	885.56	779.88	508.38	856.41

Figure 8. Vibration modes for span length of 30 m: (a) first mode (torsion); (b) second mode (flexure).

Figure 9. Vibration modes for span length of 35 m: (a) first mode (flexure); (b) second mode (torsion).

2.3.2. Dynamic Stability Check

This section analyzes the vertical acceleration (Figures 10a, 11a, 12a, 13a), vertical deflection (Figures 10b, 11b, 12b, 13b) and twist (Figures 10c, 11c, 12c, 13c) responses of the bridge deck obtained by moving load analysis. As a matter of fact, the largest vertical acceleration and deflection responses occur at mid-span. Besides, the maximum twist occurs at the position of the rails at spacing of 3 m in the longitudinal direction when the rails are distant by 1.5 m as suggested by the Railway Design Code. The governing response related to the vertical acceleration is seen to happen nearby the critical speed or the sub-critical speed corresponding to the first flexural mode. On the other hand, the maximum vertical acceleration in the 35 m-bridge under HEMU is observed at 390.0 km/h between the speed of 374.27 km/h corresponding to the first flexural mode and the speed of 409.16 km/h corresponding to the first torsional mode. This result seems to indicate superposition of the response due to the proximity of the flexural and torsional modes. Nevertheless, the vertical acceleration response in the analyzed bridges according to the speed of the considered trains satisfied the design limits. The vertical displacement response appears to be mostly governed by the freight train regardless of the span length of the bridge. Moreover, the response with respect to the speed per type of train is observed to be the largest at the critical speed or the sub-critical speed corresponding to the first flexural mode. Here also, the vertical displacement response in the analyzed bridges according to the speed of the considered trains satisfied the design limits. Furthermore, the twist response seems also to be mostly governed by the freight train regardless of the span length of the bridge and satisfied the design limits.

Figure 10. Moving load analysis results for span length of 30 m: (a) vertical acceleration response; (b) vertical displacement response; and (c) twist response.

Figure 11. Moving load analysis results for span length of 35 m: (a) vertical acceleration response; (b) vertical displacement response; and (c) twist response.

Figure 12. Moving load analysis results for span length of 40 m: (a) vertical acceleration response; (b) vertical displacement response; and (c) twist response.

Figure 13. Moving load analysis results for span length of 45 m: (a) vertical acceleration response; (b) vertical displacement response; and (c) twist response.

3. Performance Evaluation on Full-Scale PSC I-girder Specimen

3.1. Full-Scale PSC I-girder Specimen

A full-scale specimen of the PSC I-girder was fabricated to check the validity of the values applied for the damping ratio and the natural frequencies as governing factors on the amplitude of the dynamic responses. In addition to the dynamic test, static test was also performed to evaluate the load bearing capacity of the girder. The specimen of which dimensions are shown in Figure 14 was fabricated to span over 35 m among the 4 considered span lengths. Note that cuttings were done in the bottom of the girder at a longitudinal distance of 1100 mm from the ends to prevent the occurrence of overturning during the transportation and construction process of the PSC I-girder. The height of the girder including the 140 mm of the half-deck is 2.5 m. The concretes used for the specimen apply the values listed in Table 2, which is a compressive strength of 50 MPa for the girder and 30 MPa for the deck. The specimen was fabricated in 3 segments (12 + 10.9 + 12 m) aligned longitudinally and assembled by post-tensioning. Finally, a monolithic structure was obtained by placing the deck concrete.

Figure 14. Shape of 35 m-long full-scale PSC I-girder specimen: (a) longitudinal view with 3 segments (12 + 10.9 + 12 m); (b) sectional view (A-A); and (c) sectional view (B-B).

3.2. Evaluation of Dynamic Characteristics

The dynamic test was conducted to verify the validity of the analytical model in Section 2. The natural frequencies of the specimen were compared to the values obtained by moving load analysis. Forced vibration was applied using the exciter shown Figure 15a and by impact through manpower as shown in Figure 15b.

Figure 15. Forced vibration methods: (a) exciter; (b) manpower.

For the vibration caused by the exciter, the frequency of the forcing excitation was increased stepwise and the behavior of the specimen was observed. Figure 16a plots the acceleration measured at the bottom of the girder. The largest amplitude of the acceleration occurred under excitation frequencies between 4.2 Hz and 4.4 Hz. After having roughly verified the range of the natural frequency of the specimen, impact test was conducted by two men jumping on the girder as shown in Figure 15b to generate free vibration and check the natural frequency. FFT (Fast Fourier Transform) analysis was then performed on the vertical acceleration data acquired during the free vibration of the girder as shown in Figure 16b. The so-obtained natural frequency was 4.2 Hz, which agreed fairly with the analytic value of 4.27 Hz for the span length of 35 m. Figure 16c plots the residual vertical displacement from which a damping ratio higher than 1% was estimated by curve fitting. Accordingly, the value of 1% adopted in the dynamic analysis can be assumed to be valid for the conservative design of the PSC I-girder railway bridge.

(a)

Figure 16. *Cont.*

(b)

(c)

Figure 16. Dynamic characteristics of PSC I-girder specimen: (a) acceleration response under forced vibration by exciter; (b) FFT analysis; and (c) free vibration for analysis of damping ratio.

3.3. Evaluation of Ultimate Load Bearing Capacity

3.3.1. Test Method

Four-point bending test was carried out on the specimen to evaluate its structural performance. Figure 17 illustrates the four-point bending test setup. The specimen was placed on two supporting pins positioned at a distance of 500 mm from the ends. Two loading actuators were placed at an equal 3 m apart from center of specimen, which let this distance be subjected to pure bending moment. Two actuators with capacity of 3500 kN were used to apply loading and, the deflection, the strains in concrete and steel as well as the crack width at the segmental interfaces were measured. The test was planned to apply loading from the service load to the ultimate load and up to failure but failure could not be attained due to the limited capacity of the actuators and the maximum load reached 6097 kN.

Figure 17. Static loading test of PSC I-girder: (a) 4-point bending test set up; (b) photograph of actual test.

3.3.2. Test Results

Figure 18 shows the crack map of the specimen at the maximum load of 6097 kN. Cracking initiated at the bottom in the center of the girder at 2530 kN. In view of the cracking pattern, the specimen exhibited typical flexural behavior until the maximum load.

Figure 18. Crack map of specimen at maximum load.

The structural performance of the specimen was evaluated by comparing the experimental values with the load and deflection at ultimate limit state caused by the live load specified in the design code. Figure 19a,b plot respectively the deflection at mid-span and the crack width according to the increase of the load. Table 6 arranges the flexural performance requirements at service limit state and ultimate limit state provided by the design code. Note that the flexural performance here relates only to the live load. Table 6 also summarizes the amplitude of the load corresponding to each limit state and the corresponding vertical deflection measured in the specimen. The amplitude of the load at service limit state and ultimate limit state was calculated based upon the flexural strength deduced from the design code and the distance from the support to the loading point (13.95 m). As a result, the vertical deflection measured at the maximum load (6097 kN) was 361 mm, which provided vertical deflection of 43 mm for the ultimate limit state (2150 kN) and 21.9 mm for the service limit state (1162 kN).

Figure 19. Results of static test of specimen: (a) load-displacement curve; (b) load-crack width curve.

Table 6. Section forces and loads according to limit states.

Maximum Load at Experiment (kN)	Service Limit State			Ultimate Limit State		
	Flexural Strength (kN-m)	Load (kN)	Deflection (mm)	Flexural Strength (kN-m)	Load (kN)	Deflection (mm)
6097	8105	1162	21.9	14,995	2150	43

Even if the static test could not be conducted until failure and was terminated at the maximum load of 6097 kN due to the limited capacity of the actuators, the specimen appears to secure sufficient safety factor considering the loads at service limit state and ultimate limit state. The corresponding vertical deflections also satisfied sufficiently the design limit ($L/600 = 56.5$ mm). Moreover, the crack width measured at the segment interfaces at each limit state fell below the allowable crack width of 0.2 mm specified in the design code. Consequently, the segmentally fabricated specimen showed monolithic behavior.

4. Conclusions

This study presents the yearly evolution of bridges, especially PSC I-girder bridges in Korea and investigates analytically and experimentally the performance of the new PSC I-girder bridge to be applied as railway bridges. The following conclusions can be drawn.

- (1) Natural frequency analysis for the new PSC I-girder range from a span of 30 to 45 m revealed that their first and second mode occur very close, either in bending or torsion mode alternately.
- (2) Subsequent moving load analysis was conducted considering 5 types of trains in Korea. The new type of PSC I girder is likely to amplify the dynamic response due to the proximity of the first two modes, but it was found that it satisfied sufficiently all the design limits related to the acceleration, deflection and twist.
- (3) Experimental verification of natural frequency was carried out on a full-scale specimen with span length of 35 m. The natural frequency (4.2 Hz) of the specimen extracted from the dynamic test results was in good agreement with the value (4.27 Hz) obtained analytically and validated the dynamic analysis model established in this study.
- (4) The performance of the specimen observed in the static test secured sufficient safety through the comparison of the load and deflection corresponding to the service and ultimate limit states of the design code. Accordingly, the new PSC I-girder presented in this study develops the dynamic performance and load bearing capacity required for its application in railway bridges and can be applied in new constructions under plan in Korea.

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