SemantJournals

American Journal of Technology Advancement 2997-9382, Vol 03, No. 06, 2026

Article

Structural Assessment of Highway Bridges through Dynamic Load Testing: A Comparative Study of Field Measurements and Finite Element Simulations

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Abstract: The current study assesses the in-service dynamic performance of three PSC moveable bridges (four main spans, overall length ≈ 66.5 m, width ≈ 9.4 m) located on the Hilla–Kish Expressway, Iraq. IMU/gyroscopic sensors used for controlled vehicle passage field dynamic load testing, baseline (no-traffic) and moving-load records were processed to obtain natural frequency (FFT), traffic-induced vibration frequency, and dynamic displacement (double integration with filtering). A three-dimensional finite element model corresponding to the bridge superstructure was developed in CSI Bridge (ver. 26) in order to produce the same numerical signals at their joints. Agreement was quantified using an efficiency coefficient E_c(experimental/theoretical). Frequency-based results showed $E_c < 1$(natural: 0.960–0.995; moving-load vibration: 0.936–0.958), indicating Poor agreement, whereas dynamic displacements showed $E_c > 1$(upward ≈1.084–1.086; downward ≈1.073–1.087). The Poor frequency agreement is attributed to the as-built condition, notably the absence of diaphragms and a pier cap, which typically increase transverse connectivity and stiffness near piers, highlighting the importance of capturing construction details in dynamic field–model validation.

Keywords: Bridge Assessment, Dynamic Testing, Vibration, Downward, Finite Element Model

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1. Introduction

A bridge is a structure that is used to give passage, protection to the cars / people (and sometimes the utilities) through the obstacles, such as waterway, valley, and road, and could be crossed without blocking what is in the top or bottom. Bridge systems are often defined in terms of superstructure elements (deck, girders or beams, bearings, joints, pavement layers, barriers, drainage, etc.) and substructure elements (piers, caps, abutments, foundations)[1]. Bridge selection depends on site constraints (geotechnical/hydraulic), geometry/profile, cost, and traffic demand. ridge categories are often defined by material (prestressed concrete, steel, etc.) and by support system (simply supported vs continuous)[2].

Bridges act as important links in transportation networks which support economic activity

and social interaction; as economic development and network extension occur, the demand for bridges increases. Reliability performance evaluation is of utmost importance for maintaining public safety and service, as many bridges have been subjected to overloading and environmental effects. In addition, long-term performance data are often sparse for precast systems and rapid bridge construction, emphasizing the need for monitoring and testing. [3][4][5][6].

Typical bridge assessment combines damage surveys with recorded structural response and analytical verification. Many agencies use periodic (e.g., biennial or greater) inspections for damage investigation (DI) to validate safety, identify maintenance/repair/strengthening needs, and to aid with funding/planning decisions, where in this context the DI falls into periodic cyclic inspections. Load tests are usually static and dynamic tests, and static tests commonly focus on deflection/strain/stress at critical locations (midspan/quarter-span [2]). Field-validated "efficiency/validation" factors are also used to assess agreement of measured and theoretical responses and infer stiffness/safety margins [7][8][9][10][11][12].

Maintenance actions are determined directly from inspections, especially for the repeated vulnerable components (drainage, expansion joints, bearings and protecting systems). As an example, inspection of a steel I-girder span highlighted clearing blockages, repair of the drainage apparatus, high humidity-proof coating, and repair or substitution of extension joints as a basis of major distress. Monitoring outputs may also guide maintenance strategies, allowing for “pre-emptive” interventions before the damage becomes severe. For such bridges with dynamic issues, desire to improve stiffness and reduce long-duration vibration response (i.e. strengthen connections/cross-beams) could also be recommended[13][14][15][16][17][18][19] [20].

Dynamic tests, conducted after static tests that confirm satisfactory elastic behavior, are then used to quantify vibration-sensitive behavior under traffic. A practical dynamic-static load (DSL) framework is then developed which utilizes strain sensors and deflection measurement systems to measure static loading (to determine stress state) and dynamic properties (under moving loads). Field programs also evaluate loading efficiency, staging, and process control so that they do not create damage that would mask its diagnostic value. The state-of-the-art bridge deck in-situ testing has been conducted with embedded sensor networks (e.g., vibrating-wire strain gauges) and optical surveying to provide real-time equivalent stresses and full-fied response “surfaces”. Findings demonstrate that built-in sensors provide more precise/accurate options for conventional optical beam deflection readouts, yet can still allow analytical verification [21][22][23][24][25][26][27].

The use of numerical finite element analysis (FEA/FEM) to model complex bridge behavior, minimize the reliance on simplified hand methods, and aid in assessment beneath static and dynamic loading... The modelling traffic loads (HS20-44) and service actions (permanent, temperature, wearing surface, wind, etc. CSI-Bridge was used for investigation of the static and dynamic responses under moving and combination loads of the curved box girder bridges, to allowing a code-based serviceability verification. Modeling also extends to time-history integration for use in dynamic performance studies to potentially compare natural frequency (zero traffic) against forced frequency under vehicle passage to infer stiffness adequacy (Noiz factor). [28][29][30][31][32][33][34][35].

The commonly used evaluation philosophy is to compare measured bridge response with the theoretical response under the same test loads, as bridges in reality usually display a higher capacity than the simplified theory: unintended composite action, load distribution, parapet participation, unintended continuity, and span shape effects are frequently observed. The field programs thus focus on consistency checks: deflections are used for deriving stresses, and they are then compared to stresses based on sensor-induced strains for verification. Such that in the case of DSL testing, the validation/efficiency coefficient (measured/theoretical) is seen as a performance metric — smaller values indicating that results match better and greater margin of stiffness [2][36][37] [38][39] [40].

DESCRIPTION OF THE BRIDGE STRUCTURE

The bridges that were studied in this work are In-Service Prestressed Concrete overpass girder bridges which are located along the Hilla–Kish Expressway in Iraq. The total length of the bridges is approximately 66.5 m, with 4 spans and having 9.4 m with (7.0 m-carriageway and 1.2 m each side sidewalks) in width. The deck system consists of a continuous riding surface of a reinforced concrete slab cast over precast girders, with an asphalt wearing course. The superstructure is made of prestressed U-girders (internal long girders and external short girders)—generally, there are four internal and four external girders per span—with typical girders spans of around 23.0 m and 10.25 m, respectively. Elastomeric bearings are placed at the supports to transfer vertical loads and release rotations. Multi-column reinforced concrete bents (approximately four columns per bent), which forms part of the substructure with a height of 6.75 m from the top of the foundation to the bottom of the superstructure Also, the bridge approaches are supported on reinforced concrete abutments with wing walls. Pilecap: Foundation system consists of reinforced concrete pile caps, resting on piles of 1.2 m nominal and diameter pile. Fig. 1 shows the longitudinal section of the bridge.

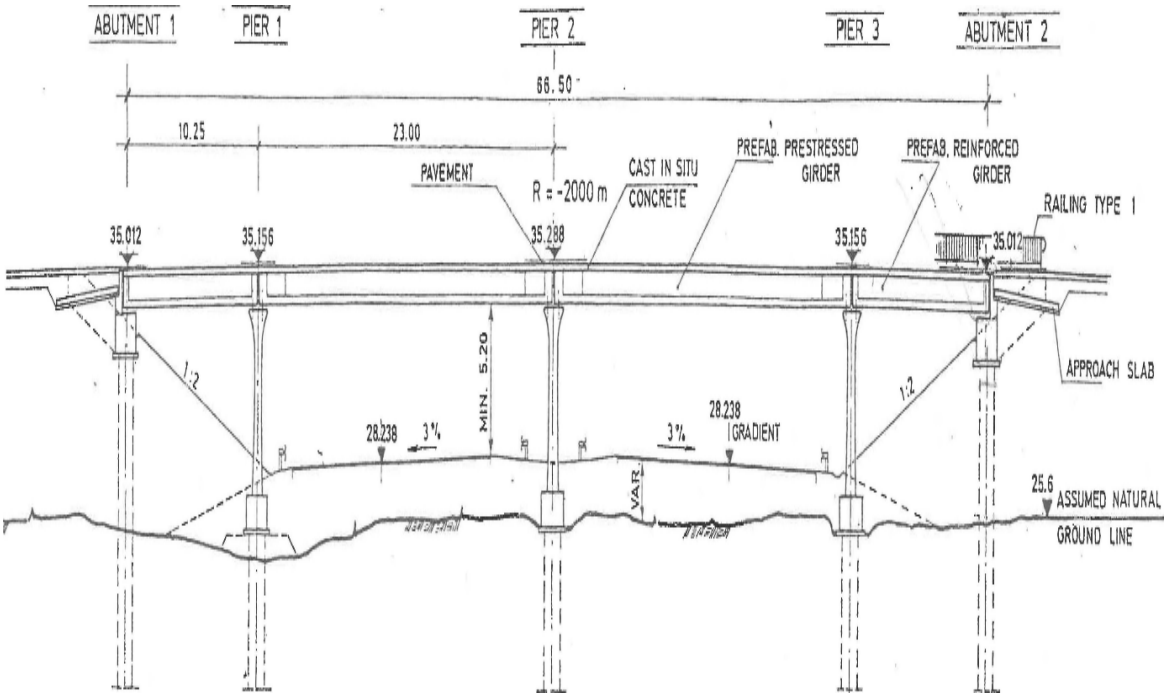


Fig. 1. Longitudinal section of the bridge.

FIELD DYNAMIC LOAD TESTING

The in-service vibration characteristics and dynamic serviceability of the investigated prestressed concrete (PSC) girder bridges were evaluated thru controlled moving-load excitation dynamic load test. The bridge test was performed on the basis of monitoring bridge response at chosen critical locations during the passage of vehicles; the field signals recorded during the tests were subsequently used to derive pragmatic dynamic parameters for structural assessment. To enable high precision IMU/gyroscopic sensor monitoring of bridge behaviour and the nearly continuous recording of accelerations and motion histories. The testing program was implemented in three main stages: (1) baseline measurements under unloaded/no-traffic conditions to establish the reference state and identify the bridge's natural vibration characteristics; (2) application of moving loads by driving the test vehicles across the bridge at controlled speeds (from low to higher levels) to excite the structure and capture speed-dependent dynamic response; and (3)

signal processing to isolate load-induced effects by subtracting/contrasting baseline records with loaded records, then extracting key indicators including natural frequency, traffic-induced vibration frequency, and dynamic displacement. Natural frequencies were identified from free-vibration records using FFT-based spectral analysis, while vibration frequencies under moving loads were obtained using time-domain cycle identification (e.g., zero-crossing) and/or spectral methods. Dynamic displacement was estimated through double integration of acceleration with appropriate filtering to reduce drift and improve stability. Finally, the measured dynamic indicators were compared with the corresponding numerical results from the finite element model (CSI-Bridge) to verify consistency between measured and simulated in-service behaviour and to judge the adequacy of the model for representing bridge performance. Fig. 2 shows the dynamic load test configuration



Fig. 2. The dynamic load test configuration.

2. Methodology

This study used an experimental–numerical procedure to investigate the structural behaviour of three in-service prestressed concrete (PSC) highway bridges on the Hilla–Kish Expressway, Iraq. This research started with a careful evaluation of the bridge geometry, structural type, and boundary conditions to provide a foundation for both field experiments and numerical modeling. Dynamic load testing using IMU and gyroscopic sensors at chosen critical locations along the bridge spans. First, baseline measures were taken before traffic to ascertain the natural vibration properties of the structures. Then, controlled vehicle passes with specified speeds were performed to create moving-load excitations and acquire the dynamic responses of bridges. We applied filtering approaches to the recorded acceleration signals based on empirical evidence which was found to mitigate noise and drift effects. Natural frequencies were obtained from the FFT analysis; traffic-induced vibration frequencies were extracted from the response histories by spectral and time-domain analyses. Dynamic displacements were derived from the measured accelerations by performing double integration with associated correction procedures. A 3D finite element model of the bridge system was created using CSI Bridge Version 26 to confirm the experimental results. The actual bridge geometry, material properties, boundary conditions and loading scenarios were used in the numerical model to represent vehicle-excited dynamic behavior. Theoretical values of natural frequency, vibration frequency and dynamic displacement were obtained using modal and dynamic analyses. As for the last stage, all experimental and numerical results were compared through efficiency coefficients defined as the ratio of measured to predicted responses, to evaluate how similar both configurations behaved and whether or not it

characterized adequately the structural performance of the bridges under consideration.

3. Result

EXPERIMENTAL DYNAMIC LOAD TESTING RESULTS

NATURAL FREQUENCY RESULTS

The natural frequency test results of the three bridges are shown in Table 1

Table 1. Natural frequency test results.

Mode No.	Bridge 1 (Hz)	Bridge 2 (Hz)	Bridge 3 (Hz)
1	3.89	4.18	4.14
2	5.26	5.43	4.68
3	5.46	6.14	4.81
4	6.02	6.21	5.70
5	7.19	7.74	7.66
6	7.46	7.94	7.93
7	7.87	7.95	8.03
8	8.62	8.98	9.76
9	13.16	13.43	13.39
10	13.70	13.84	13.62
11	13.70	13.98	14.26
12	14.29	14.73	14.27
Average	8.89	9.21	9.02

VIBRATION FREQUENCY RESULTS

The vibration frequency test results of the three bridges are shown in Table 2

Table 2. Vibration frequency test results.

Bridge No.	Span No.	Joint No.	Longitudinal Distance	Transverse Distance	Vibration Frequency (Hz)
Bridge 1		119	5	0	7.38
	Span1	126	5	-3.068	7.65
		111	5	3.068	8.20
		528	21.5	0	4.24
	Span2	534	21.5	-3.068	4.30
		519	21.5	3.068	4.30
		1037	44.5	0	6.68
	Span3	1045	44.5	-3.068	6.83
		1029	44.5	3.068	7.20
		1445	61	0	7.96
	Span4	1454	61	-3.068	8.04
		1437	61	3.068	7.96
	Average				6.73
Bridge 2		119	5	0	7.88
	Span1	126	5	-3.068	7.73
		111	5	3.068	7.70
		528	21.5	0	4.42
	Span2	534	21.5	-3.068	4.25
		519	21.5	3.068	4.39
		1037	44.5	0	7.05
	Span3	1045	44.5	-3.068	6.98
		1029	44.5	3.068	6.90
		1445	61	0	8.13
	Span4	1454	61	-3.068	7.88
		1437	61	3.068	7.96
	Average				6.77
Bridge 3		119	5	0	8.04
	Span1	126	5	-3.068	7.98
		111	5	3.068	8.04
		528	21.5	0	4.42
	Span2	534	21.5	-3.068	4.34
		519	21.5	3.068	4.39
		1037	44.5	0	7.05
	Span3	1045	44.5	-3.068	6.98
		1029	44.5	3.068	7.13
		1445	61	0	8.13
	Span4	1454	61	-3.068	8.04
		1437	61	3.068	8.13
	Average				6.89

DYNAMIC DISPLACEMENT RESULTS

The dynamic displacement test results of the three bridges are shown in Table 3

Table 3. Dynamic displacement test results.

Bridge No.	Span No.	Joint No.	Longitudinal Distance	Transverse Distance	Upward Dynamic Displacement (mm)	Downward Dynamic Displacement (mm)
Bridge 1		119	5	0	0.109	-0.441
	Span1	126	5	-3.068	0.126	-0.210
		111	5	3.068	0.147	-0.173
		528	21.5	0	3.132	-6.567
	Span2	534	21.5	-3.068	3.089	-5.921
		519	21.5	3.068	3.120	-5.815
		1037	44.5	0	3.248	-6.625
	Span3	1045	44.5	-3.068	3.250	-5.830
		1029	44.5	3.068	3.329	-5.750
		1445	61	0	0.066	-0.429
	Span4	1454	61	-3.068	0.079	-0.130
		1437	61	3.068	0.082	-0.132
Bridge 2		119	5	0	0.110	-0.436
	Span1	126	5	-3.068	0.129	-0.210
		111	5	3.068	0.146	-0.173
		528	21.5	0	3.132	-6.560
	Span2	534	21.5	-3.068	3.118	-6.033
		519	21.5	3.068	3.149	-5.698
		1037	44.5	0	3.278	-6.674
	Span3	1045	44.5	-3.068	3.219	-5.852
		1029	44.5	3.068	3.329	-5.809
		1445	61	0	0.067	-0.433
	Span4	1454	61	-3.068	0.078	-0.130
		1437	61	3.068	0.081	-0.131
Bridge 3		119	5	0	0.109	-0.438
	Span1	126	5	-3.068	0.128	-0.208
		111	5	3.068	0.145	-0.174
		528	21.5	0	3.109	-6.591
	Span2	534	21.5	-3.068	3.144	-6.078
		519	21.5	3.068	3.175	-5.719
		1037	44.5	0	3.275	-6.588
	Span3	1045	44.5	-3.068	3.256	-5.879
		1029	44.5	3.068	3.323	-5.836
		1445	61	0	0.067	-0.436
	Span4	1454	61	-3.068	0.079	-0.130
		1437	61	3.068	0.083	-0.132

NUMERICAL DYNAMIC LOAD MODELLING

The dynamic response of the Hilla–Kish Highway bridges was simulated using a three-dimensional numerical finite element model developed in CSI Bridge (ver. 26). The model was prepared to represent the bridge geometry, structural layout, material properties, and support conditions in a manner consistent with the in-service behaviour of the multi-span prestressed concrete girder system. Dynamic loading was introduced through vehicle actions to reproduce the time-dependent structural response under traffic excitation and to capture the vibration-sensitive behaviour of the bridge during operation.

The numerical dynamic program was implemented through modal analysis to identify the bridge natural frequencies and mode shapes, followed by evaluation of traffic-induced vibration frequency and dynamic displacement at selected joints along the spans and across the bridge width. These locations were chosen to represent critical response positions and to enable a consistent interpretation of response variation along the bridge. The resulting numerical dynamic indicators (frequencies and dynamic displacements) were then used as the theoretical benchmark for later comparison with field measurements and for judging whether the simulated dynamic performance remains representative of service conditions for prestressed girder bridges. Bridge modelling is illustrated as shown. Fig. 3

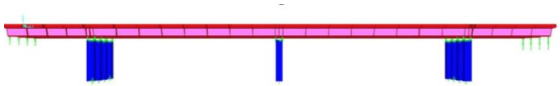


Fig. 3. The bridge modelling.

NUMERICAL STATIC LOAD TESTING RESULTS

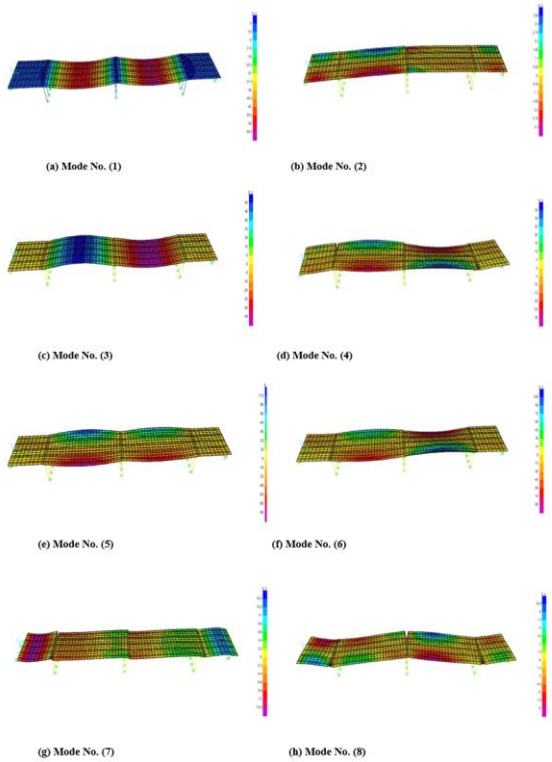
NATURAL FREQUENCY RESULTS

The natural frequency test results of the modelling bridge are shown in Table 4

Table 4. Natural frequency test results.

Mode No.	Time (sec)	Natural Frequency (Hz)
1	0.322	3.10
2	0.230	4.34
3	0.223	4.47
4	0.154	6.47
5	0.133	7.51
6	0.129	7.74
7	0.114	8.75
8	0.107	9.33
9	0.068	14.58
10	0.067	14.82
11	0.067	14.91
12	0.066	15.04
Average		**9.255**

Also, . Fig. 4 shows the values of the natural frequency due to modal analysis



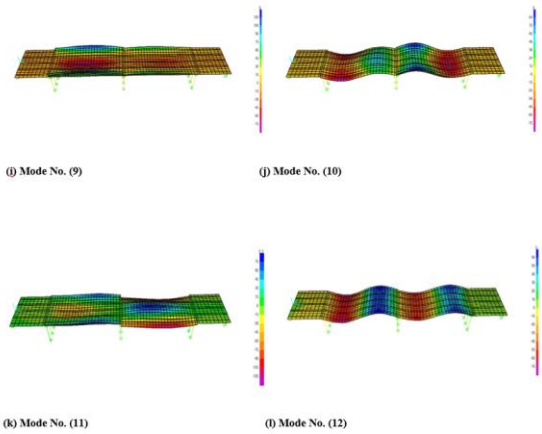


Fig. 4. Values of the natural frequency due to modal analysis.

VIBRATION FREQUENCY RESULTS

The vibration frequency test results of the modelling bridge are shown in Table 5

Table 5 Vibration frequency test results.

Span No.	Joint No.	Longitudinal Distance	Transverse Distance	Vibration Frequency (H
	119	5	0	8.29
Span1	126	5	-3.068	8.31
	111	5	+3.068	8.37
	528	21.5	0	4.56
Span2	534	21.5	-3.068	4.57
	519	21.5	+3.068	4.57
	1037	44.5	0	7.5
Span3	1045	44.5	-3.068	7.5
	1029	44.5	+3.068	7.5
	1445	61	0	8.38
Span4	1454	61	-3.068	8.38
	1437	61	+3.068	8.38
Average				7.19

DYNAMIC DISPLACEMENT RESULTS

The dynamic displacement test results of the modelling bridge are shown in Table 6

Table 6 dynamic displacement test results.

Span No.	Joint No.	Longitudinal Distance	Transverse Distance	Upward Dynamic Displacement (mm)	Downward Dynamic Displacemen (mm)
	119	5	0	0.101	-0.4047
Span 1	126	5	-3.068	0.1181	-0.1927
	111	5	+3.068	0.1353	-0.1599
	528	21.5	0	2.873	-6.137
Span 2	534	21.5	-3.068	2.914	-5.586
	519	21.5	+3.068	2.943	-5.335
	1037	44.5	0	3.064	-6.134
Span 3	1045	44.5	-3.068	3.037	-5.449
	1029	44.5	+3.068	3.054	-5.374
	1445	61	0	0.0614	-0.4014
Span 4	1454	61	-3.068	0.0726	-0.1201
	1437	61	+3.068	0.0758	-0.1211

The efficiency coefficient (E_c) was used to quantify the agreement between the experimental response (field measurements) and the theoretical/numerical response (the model), and it is defined as the ratio of the experimental value to the theoretical value. This coefficient is an important indicator in bridge assessment when comparing test results with numerical simulations to judge how closely the actual behavior matches the expected response.

For the dynamic response, the efficiency coefficients associated with the natural frequency were below unity (Bridge 1–3: 0.960, 0.995, 0.974). Similarly, the values were also below 1.0 for the vibration frequency under moving load (Bridge 1–3: 0.936, 0.941, 0.958). Since $E_c < 1$in this case, the measured dynamic frequency response was smaller than the theoretical prediction; therefore, the agreement level can be described as Poor based on the adopted dynamic load assessment classification (within the range 0.9–1.0).

In contrast, the dynamic displacement results showed efficiency coefficients greater than 1 for both the maximum upward displacement (about 1.084–1.086) and the maximum downward displacement (about 1.073–1.087), indicating that the measured displacement response was higher than the model estimates in this case. Accordingly, the interpretation of the efficiency coefficient under dynamic loading should be presented separately for each response indicator: frequencies yielded $E_c < 1$(Poor agreement), whereas displacements yielded $E_c > 1$, reflecting different matching trends between the measurements and the numerical model depending on the response type.

4. Conclusion

The dynamic assessment results indicate that the agreement level was classified as Poor because the measured frequency-based response was lower than the theoretical prediction (i.e., $E_c < 1$). This weak agreement is attributed primarily to the actual structural condition of the bridge, rather than modeling idealizations. In typical bridge construction, diaphragms and a pier cap are usually installed to provide transverse connectivity between girders, improve load distribution, and increase the overall stiffness—particularly near supports and pier regions. The absence of these members reduced the effective stiffness and altered the force-transfer path and restraint conditions at the piers, which directly affected the dynamic characteristics (especially the frequencies). Consequently, the efficiency coefficient fell into the Poor classification.

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