Analysis of Road Drainage Capacity of Developing Industrial Area in Karawang

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Abstract

Karawang regency was previously known as a rice barn in West Java, but now this area has become one of Indonesia's largest industrial districts. Its great potential increases the community's demand for business places. Therefore, the high activity of an area must be balanced with the fulfillment of infrastructure facilities. In maintaining adequate road conditions, it is necessary to provide an integrated drainage system between the road drainage system and the drainage system of the surrounding area. The objective of this study is to analyze the capacity and performance of existing road drainage channels. As the land use changes, it's crucial to evaluate the increase rainfall water runoff. By collects and evaluating recent rainfall data and hydrological analysis using the Normal, Gumbel, Log Normal, and Log Pearson III method to determine the design rainfall in South Karawang. The results of rain data processing obtained by the selected parameters test method are the Log Pearson III with 10-years of rainfall design 141.26 mm. After conducting capacity evaluation, the results show that the existing channels that serve the area (A) of 8.37 ha with the runoff coefficient (C) of 0.89 are needs to be increases to accommodate the surface rainfall water runoff discharge of, Q = 3.26 m^3/s. Thus, a precast concrete U-ditch of 1200 x 1400 mm will provide effective 400 mm of freeboard, W, and 0.5% as channel slope (s) is designed with channel discharge ($Q_{channel}$) of 3.39 m^3/s to accommodate the needs of current conditions.

Keywords: water runoff, drainage capacity, road drainage, industrial area

1. Introduction

Karawang regency is located in the northern part of West Java province, Indonesia, which is geographically located between 107° 02' - 107° 40' east longitude and 5°56' - 6°34' south latitude and has a relatively low land area with variations in elevation between 0-1.279 meters above sea level. Karawang has an area of 1,652 km^2, with a population of 2,509,839 people in 2021, and a population density of 1,312 people per km^2 [1]. Karawang regency was formerly known as a rice barn in West Java, but now this area has become Indonesia's largest industrial area. Its great potential increases the community's demand for business places. Following this rapid growth, regions that were once rice fields, wetlands, and green areas are converting into office buildings, factories, warehouses, and housing.

The high activity of an area must be balanced with the fulfilment of infrastructure facilities. Roads are a basic infrastructure needed by an area to facilitate the flow of distribution of goods and services and play a role in improving the quality of life and welfare of its people. In maintaining adequate road conditions, it is necessary to provide an integrated drainage system between the road drainage system and the drainage system of the surrounding area [2-3]. However, along with changes in land use, which causes the land surface to be covered, the water catchment area is decreasing. So, the runoff

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coefficient tends to be increased. As a result, more water flows on the surface rather than absorbed by the ground [4-5].

Some roads in Karawang regency often experience puddles during heavy rains. One of them is the Trans-Heksa road, the main road that connects six industrial areas. Flooding on this road causes several losses, including disruption to the comfort of road users, delays in mobilization, and a decrease in road quality as long-term effects. Therefore, it can be inferred that the current road drainage in the Karawang industrial area needs to be properly evaluated to control inundation or flooding and not interfere with mobilization activities on this road. This study aims to analyze the capacity of the existing road drainage channels [6-8] in the Karawang industrial area against the needs of current locations and then continue by designing road drainage channels that can accommodate the particular rainfall and runoff discharge designs in the Karawang industrial area.

2. Material and Method

The data used in this study is the annual maximum daily rainfall data that available in the report of study regarding hydrological and hydraulic analysis in the Karawang area from 2003 to 2020. Based on the literature study, various information related to the drainage system are obtained. The literature sources used include reference books, journals, reports, or related research: Road Drainage Planning Module by the Research and Development Agency of the Ministry of Public Works, reference books on hydrology, hydraulics, and drainage systems, and previous studies and papers on similar topics [9-12].

Following the completion of the hydrological analysis [13], the study will proceed to analyze the amount of runoff discharge plans by calculating the concentration time and rainfall intensity from the previously selected method. The hydraulics study started when the runoff discharge plan's amount was determined and compared to the existing drainage capacity. The Manning formula was utilized in this analysis to determine the current drainage capacity's runoff discharge. Determine whether the existing drainage system is able to accommodate the quantity of planned runoff discharge by comparing the planned runoff discharge with the current drainage capacity. In this study, the area under analysis is the south side of the Trans-Heksa road drainage channel in the Karawang industrial area, West Java, which is 706 m long, and the surrounding area, as shown in Fig. 1. The methodology of the study is shown in Fig. 2.

Fig. 1 Study location area [14]

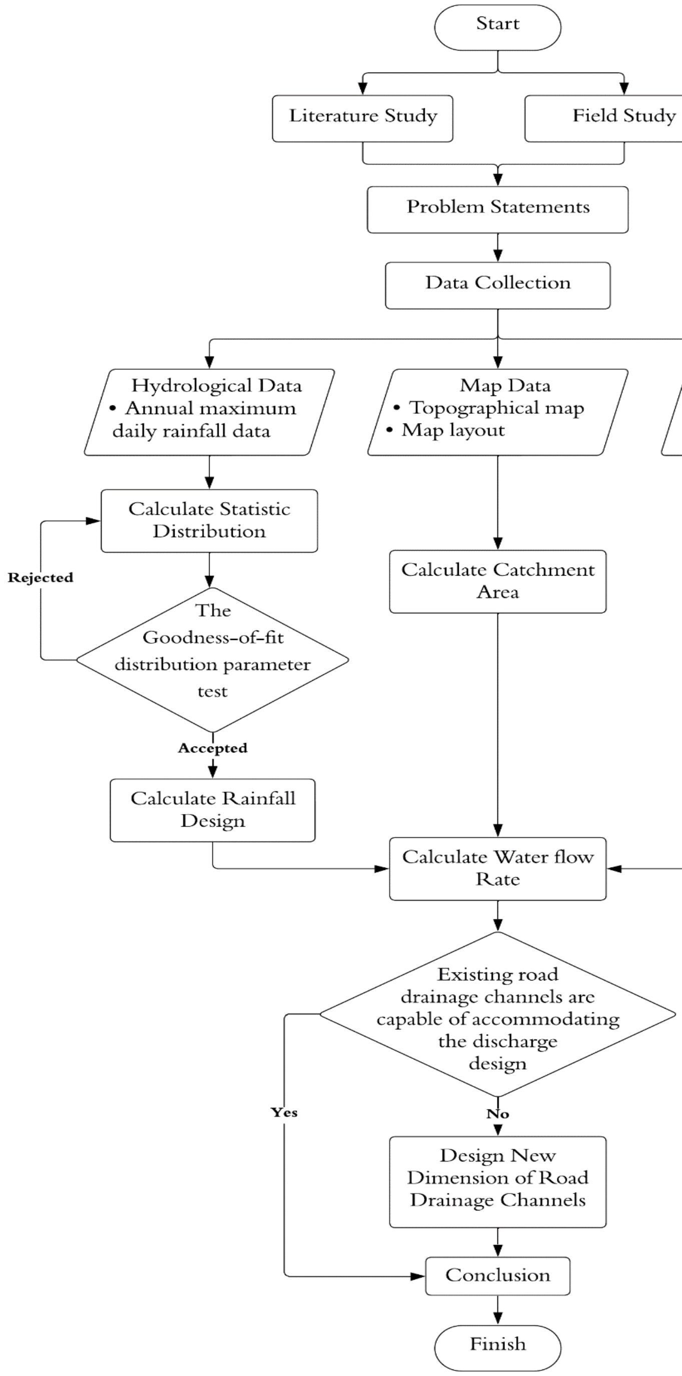

Fig. 2 Study methodology

3. Results and Discussion

3.1 Hydrological analysis

The rainfall data being analyzed in this study is from 2003 to 2020 (18 years) that was collected mainly from Meteorological Climatological and Geophysical Agency (BMKG) as well as some hidrology study reports. The summary of annual maximum daily rainfall in Karawang is shown in Table 1.

Table 1 Annual maximum daily rainfall in Karawang

Year	**R^{24} (mm)**	**Year**	**R^{24} (mm)**
2002	103.47	2012	91.31
2003	113.00	2013	110.38
2004	77.42	2014	68.56
2005	81.12	2015	41.99
2006	102.99	2016	57.01
2007	120.20	2017	55.57
2008	90.42	2018	72.92
2009	111.83	2019	70.48
2010	92.20	2020	109.16
2011	97.35		

The above rainfall data was then evaluated and calculated to obtain the statistical values and tested to determine which method is suitable for the characteristics of available data. The conclusion is shown in Table 2. It is known that the most suitable method for the available data is the Log Pearson III method.

Table 2 The Conclusion of statistical parameters computation of Karawang data

No	**Distribution**	C_s ***Condition***	C_k ***Conditio***	C_s ***Analysis***	C_k ***Analysis***	**Decision**
1	Normal	$C_s \approx 0$	$C_k \approx 3$	-0.4533	2.7017	Rejected
2	Log-Normal	$C_s = 0.1934$	$C_k = 3.0665$	-0.9554	3.7885	Rejected
3	Gumbel	$C_s \approx 1.14$	$C_k \approx 5.4$	-0.4533	2.7017	Rejected
4	Log Pearson III	$C_s = flexible$	$C_k = flexible$	-0.9554	3.7885	Accepted

After that the test of Chi-Square and Kolmogorov-Smirnov is carried out to find out that the data fulfill the requirements. It showed that the value of X^2 which is 4.12 is smaller than the X_{cr}^2 which is 5.991. Other than that, the value of D_{max} in Kolmogorov-Smirnov is also smaller than the value of D_0 (0.1410 < 0.29). Therefore, the availabe data and the Log Pearson III method can be used to calculate the rainfall design. The rainfall design of South Karawang is shown in Table 3.

Table 3 Rainfall designs in South Karawang

Return Period (years)	K	$\log X$	X (mm)
2	0.1567	1.9473	88.57
5	0.8521	2.0336	108.04
10	1.1355	2.0687	117.15
25	1.3836	2.0995	125.76

The next step is to calculate the water flow rates (Q) for return periods of 2, 5, 10, and 25 years. From the field study observation, it's known that three sections of road drainage channels serve the area of study shown in Fig. 3. The sections of service areas are divided according to the location of the existing road drainage channels. Every section area represents the catchment area of its drainage channel. The catchment area includes half of the median, the carriageway, the road shoulder, and its surrounding area.

Fig. 3 Sections of service area

For example, in section 1, the service area in section 1 includes:

a. Carriageway: L = 335.00 m; W = 7.00 m

$A_{carriageway} = L\ x\ W = 335.00\ x\ 7.00 = 2345\ m^2$

b. Median: L = 335.00 m; W = 8.65 m

$A_{median} = L\ x\ W = 335.00\ x\ 8.65 = 2897.75\ m^2$

c. Shoulder: L = 335.00 m; W = 3.00 m

$A_{shoulder} = L\ x\ W = 335.00\ x\ 3.00 = 1005.00\ m^2$

d. Surrounded area: L = 335.00 m; W = 100.00 m

$A_{surarea} = L\ x\ W = 335.00\ x\ 100.00 = 33500.00\ m^2$

Total area = $A_{carriageway} + A_{median} + A_{shoulder} + A_{surarea}$

$= 2345 + 2897.75 + 1005.00 + 33500.00$

$= 39747.75\ m^2 = 0..039748\ km^2$

There is a different runoff coefficient (*C*) for every land surface. Because there are different types of surfaces in one area, the runoff coefficient value used is the average coefficient value. For example, in section 1, the value of *C* for each zone is written below:

a. Carriageway formed by concrete = 0.95
b. Median formed by lawn = 0.3
c. Shoulder formed by lawn = 0.3
d. Surrounded Area is industrial area = 0.8 and 1.2 (*fk*)

Therefore, the average *C* for section 1 becomes:

$$C = \frac{(C_1 \cdot A_1) + (C_2 \cdot A_2) + (C_3 \cdot A_3 \cdot fk_3)}{A_1 + A_2 + A_3}$$

$$C = \frac{(0.95\ \text{x}\ 2345.00) + (0.3\ \text{x}\ 2897.75) + (0.3\ x\ 1005.00) + (0.8\ x\ 33500.00\ x\ 1.2)}{2345.00 + 2897.75 + 1005.00 + 33500.00}$$

$$C = \frac{2227.75 + 869.325 + 301.5 + 32160}{39747.75} = \frac{35558.575}{39747.75} = 0.89461$$

The calculation of rain intensity in this study uses the Mononobe formula for return periods of 2, 5, 10 and 25-years rainfall. For example, in Section 1 is known that:

$t_c = 0.158\ hours$

- 2-years return period

$$I = \frac{R^{24}}{24}\left(\frac{24}{t_c}\right)^{\frac{2}{3}}$$
$$= \frac{88.57}{24}\left(\frac{24}{0.158}\right)^{\frac{2}{3}}$$
$$= 105.062\ mm/hour$$

- 10-years return period

$$I = \frac{R^{24}}{24}\left(\frac{24}{t_c}\right)^{\frac{2}{3}}$$
$$= \frac{117.15}{24}\left(\frac{24}{0.158}\right)^{\frac{2}{3}}$$
$$= 138.963\ mm/hour$$

- 5-years return period

$$I = \frac{R^{24}}{24}\left(\frac{24}{t_c}\right)^{\frac{2}{3}}$$
$$= \frac{108.03}{24}\left(\frac{24}{0.158}\right)^{\frac{2}{3}}$$
$$= 128.145\ mm/hour$$

- 25-years return period

$$I = \frac{R^{24}}{24}\left(\frac{24}{t_c}\right)^{\frac{2}{3}}$$
$$= \frac{125.76}{24}\left(\frac{24}{0.15}\right)^{\frac{2}{3}}$$
$$= 149.177\ mm/hour$$

The rainfall water runoff discharge (Q_{need}) that have to accommodate by drainage channels in this study are calculated using the rational method. The discharge design is planned for 2, 5, 10 and 25-years return period. This calculation requires the value of the average runoff coefficient (*C*), rainfall intensity (*I*), and service area (*A*). For example, in section 1 is known that:

$$C = 0.89641$$
$$A = 0.039748\ km^2$$

- Q_{need} for 2-years return period

$$Q = \frac{1}{3.6} C\ x\ I\ x\ A$$
$$= \frac{1}{3.6} 0.89461 \text{ x } 105.062 \text{ x } 0.039748$$
$$= 1.038\ m^3/s$$

- Q_{need} for 10-years return period

$$Q = \frac{1}{3.6} C\ x\ I\ x\ A$$
$$= \frac{1}{3.6} 0.89461 \text{ x } 138.963 \text{ x } 0.039748$$
$$= 1.373\ m^3/s$$

- Q_{need} for 5-years return period

$$Q = \frac{1}{3.6} C\ x\ I\ x\ A$$
$$= \frac{1}{3.6} 0.89461 \text{ x } 128.145 \text{ x } 0.039748$$
$$= 1.266\ m^3/s$$

- Q_{need} for 25-years return period

$$Q = \frac{1}{3.6} C\ x\ I\ x\ A$$
$$= \frac{1}{3.6} 0.89461 \text{ x } 149.177 \text{ x } 0.039748$$
$$= 1.473\ m^3/s$$

The complete calculation to determine the rainfall water runoff discharge (Q_{need}) is presented Table 4.

Table 4 The Rainfall water runoff discharge design

Return Period	Water flow Design (Q_{need})				
	Zone	C	$I (mm/hour)$	$A (km^2)$	$Q (m^3/s)$
2-years	Section 1	0.89461	105.062	0.039748	1.038
	Section 2	0.89461	121.281	0.017798	0.536
	Section 3	0.89461	119.271	0.026222	0.777
5-years	Zone	C	$I (mm/hour)$	$A (km^2)$	$Q (m^3/s)$
	Section 1	0.89461	128.145	0.039748	1.266
	Section 2	0.89461	147.929	0.017798	0.654
	Section 3	0.89461	145.476	0.026222	0.948
10-years	Zone	C	$I (mm/hour)$	$A (km^2)$	$Q (m^3/s)$
	Section 1	0.89461	138.963	0.039748	1.373
	Section 2	0.89461	160.417	0.017798	0.709
	Section 3	0.89461	157.758	0.026222	1.028
25-years	Zone	C	$I (mm/hour)$	$A (km^2)$	$Q (m^3/s)$
	Section 1	0.89461	149.177	0.039748	1.473
	Section 2	0.89461	172.207	0.017798	0.762
	Section 3	0.89461	169.352	0.026222	1.104

3.2 Hydraulics analysis

Calculation of the discharge or water flow of the existing channel ($Q_{existing}$) that represent the channel capacity in this study uses the Manning formula. Channel dimensions are obtained from field study observation. For example, channel in section 1 has:

- $V = 1.76\, m/s$
- $A = 0.7\, m^2$

Therefore, the discharge of channel ($Q_{existing}$) becomes:

$$Q = V\, x\, A$$
$$= 1.76\, x\, 0.7$$
$$= 1.232\, m^3/s$$

The complete calculation of existing channel capacity can be seen in Table 5.

Table 5 Existing channel water flow

Existing Channel Water flow ($Q_{existing}$)			
Zone	$V (m/s)$	$A (m^2)$	$Q (m^3/s)$
Section 1	1.76	0.7	1.232
Section 2	1.89	0.81	1.529
Section 3	2.43	0.906	2.198

In the previous section, the calculation of the rainfall water runoff discharge design (Q_{need}) and existing channel discharge ($Q_{existing}$) has been carried out. Next, existing channels will be evaluated by comparing the $Q_{existing}$ against the Q_{need}. It aims to identify whether the existing channel is able to accommodate the appropriate discharge needs. If the existing channel is incapable of flowing the Q_{need}, theoretically, it interprets that water overflow will occur. So, the channel must be redesigned with proper dimensions, thus capable of accommodating the Q_{need}.

In this study area, the condition shows that the water flows from channel in section 1 to channel in section 2 and ends in section 3, which will then be flowed into a box culvert to be channelled into the Cidawolong river. Therefore, the channel in section 2 has to accommodate the Q_{need} of section 1 channel also, as well as the channel in section 3 besides accommodate its Q_{need}, it needs to accommodate the Q_{need} of section 2 and section 3.

For example, channel in section 2 for 2-years return period has to accommodate:

$$\begin{aligned} Q_{need}\ Section\ 2 &= Q_{need}\ Section\ 1 + Q_{need}\ Section\ 2 \\ &= 1.038\ m^3/s + 0.536\ m^3/s \\ &= 1.574\ m^3/s \end{aligned}$$

Table 6 Existing drainage channel evaluation

Return Period	Existing Drainage Channel Evaluation				
	Zone	Channel Shape	$Q_{existing}\ (m^3/s)$	$Q_{need}\ (m^3/s)$	Conclusion
2-years	Section 1	Trapezoid	1.232	1.038	Capable
	Section 2	Rectangular	1.529	1.574	Incapable
	Section 3	Trapezoid	2.198	2.351	Incapable
5-years	Zone	Channel Shape	$Q_{existing}\ (m^3/s)$	$Q_{need}\ (m^3/s)$	Conclusion
	Section 1	Trapezoid	1.232	1.266	Incapable
	Section 2	Rectangular	1.529	1.920	Incapable
	Section 3	Trapezoid	2.198	2.868	Incapable
10-years	Zone	Channel Shape	$Q_{existing}\ (m^3/s)$	$Q_{need}\ (m^3/s)$	Conclusion
	Section 1	Trapezoid	1.232	1.373	Incapable
	Section 2	Rectangular	1.529	2.082	Incapable
	Section 3	Trapezoid	2.198	3.110	Incapable
25-years	Zone	Channel Shape	$Q_{existing}\ (m^3/s)$	$Q_{need}\ (m^3/s)$	Conclusion
	Section 1	Trapezoid	1.232	1.473	Incapable
	Section 2	Rectangular	1.529	2.235	Incapable
	Section 3	Trapezoid	2.198	3.339	Incapable

Based on the evaluation in

Table 6, most of the existing channels are incapable of accommodating the Q_{need}. Therefore, new channel designs with proper dimensions are required to appropriately serve the area. Previously, it identified that the roadside channel in this study serves an area that was divided into three sections that have the same runoff coefficient (C) of 0.89461 because all of the channels providing services for the same catchment area (A) accumulated are equal to 83766.90 m^2, consisting of the road body, shoulder, median and surrounding area. Therefore, the new design will be determined as one section with the same catchment area as shown in Table 7.

Table 7 The Calculation of new drainage profile

Return Period	Channel Shape (mm)	Channel Material	Conduit Time (tf)										
			Cross – Section			$A\ (m^2)$	$p\ (m)$	$R\ (m)$	$L_s(m)$	n	s	$V\ (m/s)$	$tf\ (min)$
			$b\ (m)$	$y\ (m)$	$W\ (m)$								
2-years	U-Ditch 1000 x 1400	Concrete	1	1	0.4	1	3.00	0.333	706.00	0.013	0.0050	2.61	4.51
5-years	U-Ditch 1200 x 1200	Concrete	1.2	0.8	0.4	0.96	2.80	0.343	706.00	0.013	0.0050	2.66	4.42
10-years	U-Ditch 1200 x 1400	Concrete	1.2	1	0.4	1.2	3.20	0.375	706.00	0.013	0.0050	2.83	4.16
25-years	U-Ditch 1200 x 1400	Concrete	1.2	1	0.4	1.2	3.20	0.375	706.00	0.013	0.0050	2.83	4.16

The new drainage designs used is precast U-ditch channel (Table 8) because of several advantages, including guaranteed quality because the fabrication process is produced in a precast concrete factory that meets Indonesian national standards, the use of high-quality concrete so that the shape and thickness are simpler and more compact, and the installation process is relatively quickly so it doesn't disturb surrounding activities.

Table 8 The Capacity of new drainage channel profile

Return Period	New Drainage Channel			
	Channel Shape	$Q_{need}\ (m^3/s)$	$Q_{channel}\ (m^3/s)$	Conclusion
2-years	U-Ditch 1000 x 1400	2.00	2.61	Channel Capable
5-years	U-Ditch 1200 x 1200	2.46	2.56	Channel Capable
10-years	U-Ditch 1200 x 1400	2.71	3.39	Channel Capable
25-years	U-Ditch 1200 x 1400	2.90	3.39	Channel Capable

The new drainage channel taken for the roadside is the design with 10-years return period. Therefore, the U-ditch channel that appropriate for this is U-ditch 1200 x 1400 as shown in Fig. 4.

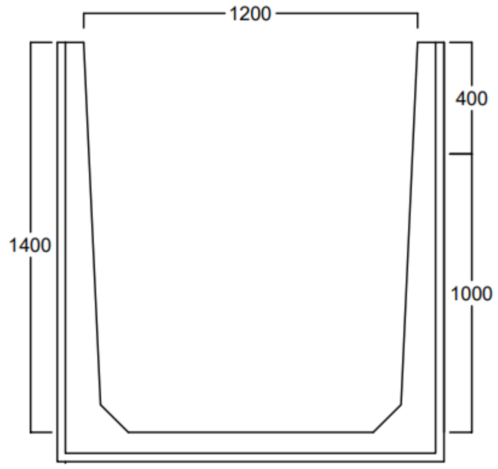

Fig. 4 U-ditch 1200 x 1400 mm.

4. Conclusions

After carrying out a series of analyses and calculations, several conclusions can be drawn from this study as follows. The rainfall design in South Karawang is obtained using the Log Pearson III method. It's known that the rainfall designs for 2-years return period is 88.57 mm, 108.04 mm for 5-years return period, 117.15 mm for 10-years return period, and 125.76 mm for 25-years return period. The channel catchment area (A) is 8.37 ha with the runoff coefficient (C) of 0.89. So then, based on the results of the hydrological and hydraulic analysis that have been carried out as an evaluation process, most all of the existing channels in section 1, section 2, and section 3 are incapable of accommodating the required water discharge for 2,5,10, and 25-years return period. Redesigning the drainage channel for 2, 5, 10, and 25-years return period have been done in order to establish road drainage channel design in the Karawang industrial area with proper capacity.

The new drainage channel of 706 m in length is designed for throughout the service area. The new drainage channel taken for the roadside is the design with 10-years return period. The design rainfall (R^{24}) of 10-years return period in the South Karawang is 117.15 mm, and the rainfall intensity (I) in the study area is 129.98 mm/hour. Therefore, the channel needs to accommodate the rainfall water runoff discharge (Q_{need}) of 2.71 m^3/s. Thus, a precast concrete U-ditch of 1200 x 1400 mm with 1000 mm as effective depth (y), 400 mm as freeboard (W), and 0.5% as channel slope (s) is designed with channel discharge ($Q_{channel}$) of 3.39 m^3/s to accommodate the needs of current conditions.

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