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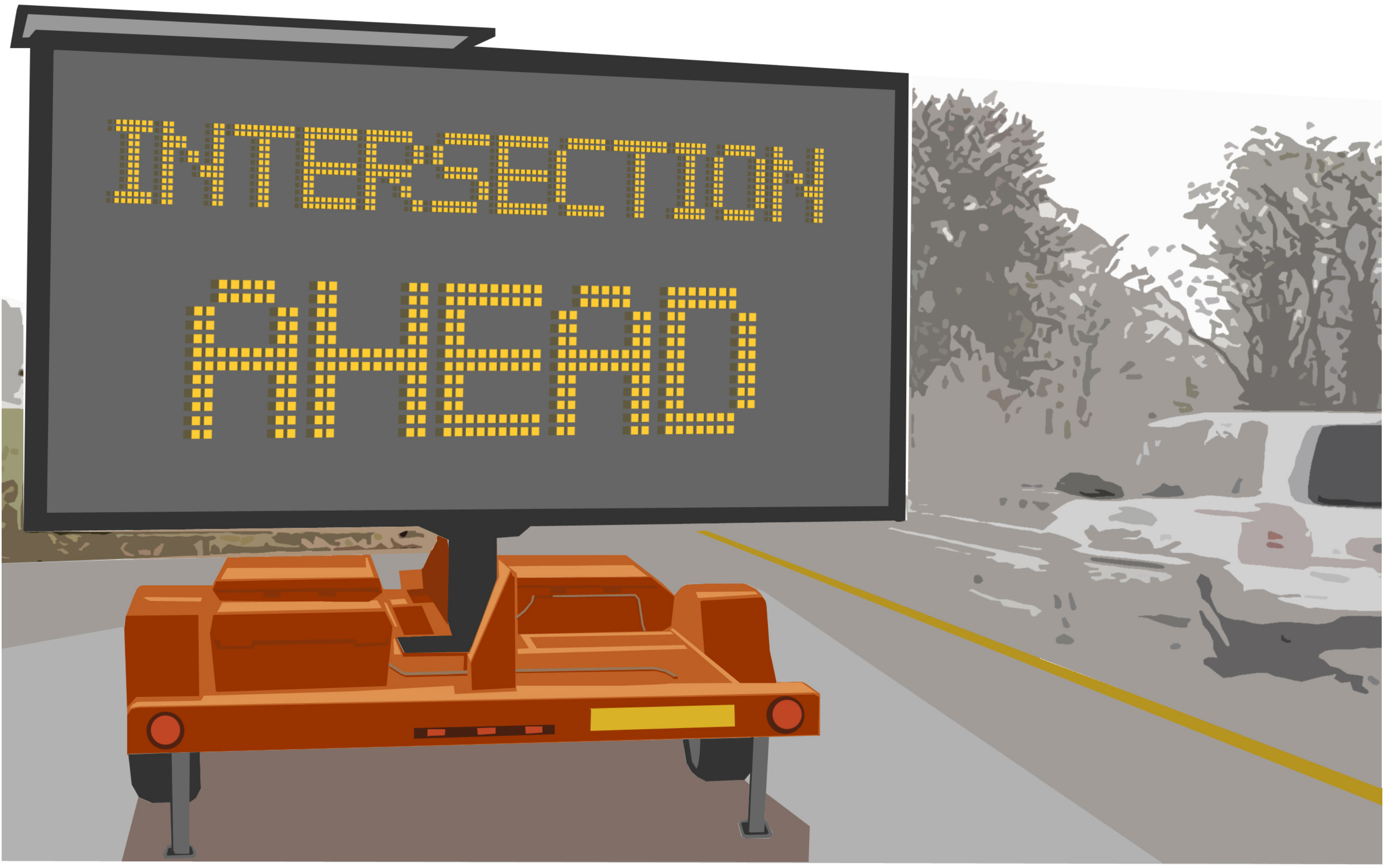

Special Issue:
The Urban Intersection

PREFACE

Dear Readers,

Season's Greetings! And Welcome to the 4th issue, volume 2, of our online-peer-reviewed International Journal of the Society of Transportation and Traffic Studies. Four issues of the journal are published annually.

This issue contains 2 academic and 2 practical papers and a special issue on a key element of road network, the intersection: The first paper present a method of how to deal with the problem of expansive clay in Bangladesh using lime stabilisation by academics from Bangladesh University of Engineering and Technology (BUET) – The second deals with trip generation model foe Bangkok high-rise residential buildings. The study made estimates of number of vehicle trip ends on weekday and weekend to help planner deal with traffic impact generated from these buildings. – The third is a practical paper from Korea addressing the driving condition and the media used to providing traffic information to drivers on their decision to change route. The goal is to promote the efficient use existing roads in order to reduce greenhouse gases and reinforce road's competitiveness in the transport sector. – The fourth, an academic paper from Thailand in which the author proposed a new enhanced combined trip distribution and traffic assignment formulation to better model traffic congestion problem.

The special issue features a practical method in addressing the problem of traffic impacts in urban areas which include congestion, accident and environmental consequences. The paper presents an introduction to the Preliminary Capacity Analysis Method (PCA-Method) which is used in the newly developed Thai Guideline for Intersection Design. A practical example is also given.

I trust you will again enjoy this issue. And from all of us at JSTS, we wish our readers a Happy New Year 2012.

Pichai Taneerananon
Professor
Chair of Editorial Board

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Special Issue

EFFECTS OF LIME STABILISATION ON ENGINEERING PROPERTIES OF AN EXPANSIVE SOIL FOR USE IN ROAD CONSTRUCTION

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ABSTRACT:

The effects of lime stabilisation on plasticity, shrinkage, swelling, moisture-density relations and strength characteristics of an expansive soil have been investigated. The soil was stabilised with lime contents of 3%, 6%, 9%, 12% and 15%. Plasticity index, volumetric shrinkage, shrinkage ratio and linear shrinkage of the treated soil decreased considerably with the increase in lime content. Free swell and free swell index of the stabilised samples decrease significantly with increasing lime content. Swelling pressure swelling potential and volume change of the treated samples also reduce markedly with the increase in lime content. With the increase in lime content, maximum dry density decreased while the optimum moisture content increased. Compared with the untreated sample, unconfined compressive strength of the treated samples increases significantly, depending on the lime content and curing age. It was found that strength development index increases with increasing curing age and lime content. It has been observed from the present investigation that long-term curing has profound influence on the gain in strength. Compared with the untreated sample, California Bearing Raito (CBR) of the stabilised samples at all levels of compaction increased significantly with increasing lime content.

KEY WORDS: expansive soil, lime stabilisation, swelling, shrinkage, unconfined compressive strength

1. INTRODUCTION

In Bangladesh, expansive soils occur in the Barind Tract of Rajshahi, Modhupur Tract, Lalmai Hill areas of Comilla, Joydevpur, Gazipur and some parts of Tangail. In the recent past, damage to buildings and other structures due to swelling of soils have been reported from different parts of Bangladesh. Foundation damages due to swelling of expansive soils can be avoided by providing proper treatment of expansive soils. Lime stabilisation has been recommended to reduce swelling of expansive soils. A number of research works (Ahmed, 1984; Rajbongshi, 1997; Molla, 1997) were carried out to investigate the geotechnical properties of lime stabilised local alluvial soils and soils from coastal regions. Investigation into properties of lime stabilised expansive soils would assess the suitability of using lime as stabiliser to reduce swelling of expansive soils. This paper presents the effect of lime stabilisation on engineering properties of a lime stabilised expansive soil.

2. SOIL USED AND LABORATORY TESTING PROGRAMME

The expansive soil was collected from Rajendrapur Cantonment, Gajipur. Disturbed and undisturbed samples were collected from the site. Liquid limit, plasticity index, shrinkage limit and linear shrinkage of the soil were found to be 56%, 43, 11% and 20%, respectively. The soil is clay of high plasticity. Free swell, free swell index, swelling pressure, swelling potential and volume change from air-dry to saturated condition of the soil have been found to be 75%, 40%, 53 kN/m^2, 1.8% and 29%, respectively. By comparing the laboratory measured values of index and swelling properties of the soil with the various recommended criteria of expansive soils, the overall degree of expansion of the soil has been found to be high.

The following testing program was undertaken for the expansive soil:

(i) Index properties, swelling properties and moisture-density relations of the untreated soil and soil treated with five different lime contents of 3%, 6%, 9%, 12% and 15% were determined.

(ii) Unconfined compressive strength test on compacted cylindrical samples of 71 mm diameter by 142 mm high tests were carried out on untreated and treated soil Unconfined compressive strength tests were carried out on lime treated samples cured at different ages of 1 week, 2 weeks, 4 weeks, 8 weeks and 16 weeks.

(iii) California Bearing Ratio (CBR) tests were carried out on untreated soil and soil treated with five different lime contents of 3%, 6%, 9%, 12% and 15%. Hossain (2001) reported details of sample preparation, procedures for various tests and equipment.

3. TEST RESULTS AND DISCUSSIONS

3.1 Effect of lime stabilisation on plasticity and shrinkage characteristics

The values of plasticity and shrinkage properties of the untreated and lime stabilised samples of the expansive soil are shown in Table 1. Compared with the untreated sample, the following effects of lime stabilisation on the plasticity and shrinkage properties of the soil were observed:

- Liquid limit of the stabilised samples initially decreased with the addition of lime content and then increased.
- Plastic limit and shrinkage limit increased significantly.
- Plasticity index, volumetric shrinkage, shrinkage ratio and linear shrinkage decreased markedly.

Plasticity indices of the treated samples were found to reduce by 30% to 56%. Ahmed (1984) found an increase in plastic limit while liquid limit and the plasticity index decreased with increasing addition of lime. Rajbongshi (1997) found an increase in plastic limit while liquid limit and the plasticity index reduced with increasing addition lime content. Holtz (1969) found that lime drastically reduces liquid limit and plasticity index of mont morillonitic clays. Compared with the untreated sample, shrinkage limit has been found to increase by 2 to 3.36 times while linear shrinkage was found to reduce by 35% to 75%. Rajbongshi (1997) also found an increase in shrinkage limit while linear shrinkage reduced with increasing addition of lime for samples of a coastal soil of Bangladesh. IRC (1976) and Bell (1993) also reported reduction in linear shrinkage with increasing lime content. Holtz (1969) found that lime drastically raises the shrinkage limit of expansive montmorillonitic clays. Compared with the untreated sample, volumetric shrinkage and shrinkage ratio were found to reduce by 48% to 82% and 31% to 46%, respectively, due to lime stabilisation.

Table 1 Comparison of Index and Shrinkage Properties of Untreated and Lime Treated Expansive Soil Samples

Index and Shrinkage Properties	Lime Content (%)					
	0	3	6	9	12	15
Liquid Limit (%)	56	54	53	54	57	59
Plastic Limit (%)	13	24	28	32	37	40
Plasticity Limit (%)	43	30	25	22	20	19
Shrinkage Limit (%)	11	22	25	29	33	37
Volumetric Shrinkage (%)	90	47	36	23	18	16
Shrinkage Ratio	2.02	1.39	1.30	1.20	1.15	1.10
Linear Shrinkage (%)	20	13	11	9	7	5

3.2 Effect of lime stabilisation on swelling properties

Table 2 presents a comparison of the swelling properties of untreated and lime stabilised samples of the expansive soil. Table 2 shows the following effects of lime stabilisation on swelling properties:

- Free swell and free swell index of the stabilised samples decreased markedly. Compared with the untreated sample, free swell and fee swell index were found to reduce by 17% to 87% and 38% to 95%, respectively, due to stabilisation with 3% to 15% lime.
- Swelling pressure and swelling potential of the treated samples reduced considerably. Swelling pressure and swelling potential become zero when the soil has been stabilised with 9% and 12% lime. These reduced swell characteristics are generally attributed to decreased affinity for water of the calcium saturated clay and the formation of a cementious matrix that resists volumetric expansion.
- Volume change of the stabilised samples from air-dry to saturated condition decreases significantly. Volume change of the stabilised samples from air-dry to saturated condition was found to decease by 55% to 83%.

The above mentioned results on the influence of lime stabilisation on swelling properties of the expansive soil clearly demonstrate that lime is an effective additive in reducing the various swelling properties of an expansive soil.

Table 2 Comparisons of Swelling Properties of Untreated and Lime Stabilised Samples

Swelling Properties	Lime Content (%)					
	0	3	6	9	12	15
Free Swell (%)	75	62	52	38	15	10
Free Swell Index (%)	40	25	17	10	4	2
Swelling Pressure of Laboratory Compacted Sample (%)	53	15	7	2	0	0
Swelling Potential of Laboratory Compacted Sample (%)	1.8	0.40	0.20	0.075	0	0
Volume Change from Air-dry to Saturated Condition (%)	29.0	13.0	10.8	7.5	6.0	5.0

3.3 Moisture-density relations of treated and untreated samples

Moisture-density relations of untreated and lime treated samples of the soil are shown in Figure 1. From the relations presented in Figure1, the maximum dry density (γ_{dmax}) and optimum moisture content (w_{opt}) of samples of the soil have been determined which are presented in Table 3. It can be seen from Table 3 that with the increase in lime content values of γ_{dmax} decreased while the values of w_{opt} increased. It was found that compared with the untreated sample, the values of γ_{dmax} decreased by 10% to 17% for an increase in lime content from 3% to 15%. The values of w_{opt} have been found to increase by 10% to 65% due to stabilisation with 3% to 15% lime.

Ahmed (1984) found that for sandy silt and silty clay soils of Bangladesh, the maximum dry density reduced with the increase in lime content. Serajuddin and Azmal (1991) also found that compared with untreated samples the maximum dry density of lime treated samples of two fine-grained regional soils decreased while optimum moisture content slightly increased. For a Chittagong coastal soil, Rajbongshi (1997) reported an increase in w_{opt} and a reduction in γ_{dmax}. Molla (1997) also found an increase in w_{opt} and a reduction in γ_{dmax} for three regional soils (liquid limit = 34 to 47, plasticity index = 9 to 26) of Bangladesh stabilised with lime.

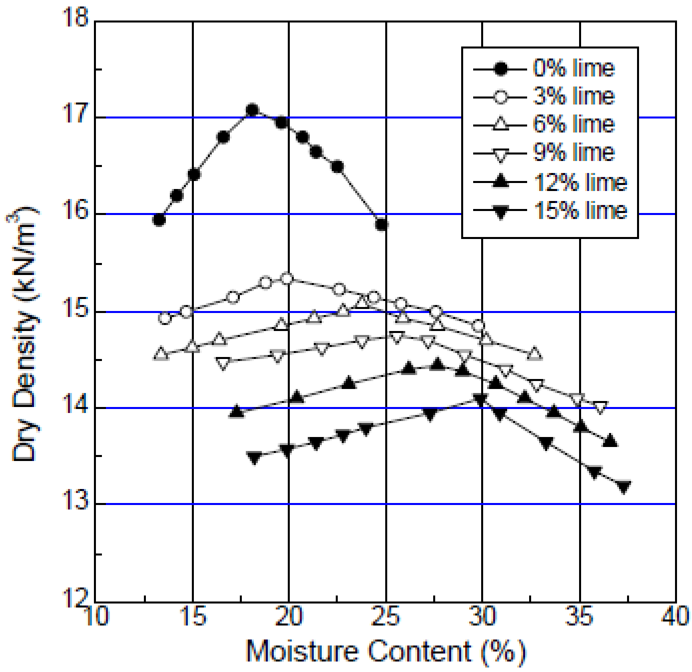

Figure 1 Moisture – density relations of untreated and lime treated samples of expansive soil

Table 3 Values of Maximum Dry – density and Optimum Moisture Content of Untreated and Lime Treated Expansive Soil

Lime Content (%)	γ_{dmax} (%)	w_{opt} (%)
0	17.08	18.10
3	15.34	19.90
6	15.08	23.80
9	14.75	25.60
12	14.44	27.70
15	14.10	29.90

3.4 Effect of lime stabilisation on unconfined compressive strength

Table 4 shows a summary of the unconfined compression test results of the treated samples of the soil. Unconfined compressive strength of the untreated sample was found to be 550 kN/m^2. Table 4 shows that compared with the untreated sample, the values of q_u of the treated samples increased significantly, depending on the lime content and curing age. It has been observed from the present investigation that long-term curing (up to 16 weeks) has marked influence on the gain in strength. It has been found that at any particular lime content, unconfined compressive strength continued to increase with the increase in curing age. The effect of long-term curing on the increase in unconfined compressive strength was found to be more pronounced when samples were stabilised with higher lime contents (more than 3%). It was found that the value of q_u of sample treated with 15% lime and cured at 16 weeks was about 8.4 times higher than the strength of the untreated sample. The relationship between q_u for different lime contents and for samples cured at different ages is shown in Figure 2. Figure 2 shows that the values of q_u of treated samples cured at any particular age increased with increasing lime content and that values of q_u of samples treated with a particular lime content increased with the increase in curing age. Serajuddin and Azmal (1991) also reported that the unconfined compressive strength of regional alluvial soils of Bangladesh treated with 5%, 7.5% and 10% slaked lime increased with the increase in lime content and curing age. Rajbongshi (1997) reported that unconfined compressive strength of lime treated samples increased with the increase in lime content and curing age for large diameter samples (71 m diameter by 142 mm high). Hasan (2002) also reported increase in unconfined compressive strength with increasing lime content for large diameter samples of silty clay collected from a reclaimed site of Dhaka city. Molla (1997) found that unconfined compressive strength of lime treated samples increased with the increase in lime content and curing age for three regional soils of Bangladesh.

The rate of strength gain with curing time has been evaluated in terms of the parameter strength development index (SDI) as proposed by Uddin (1995) and defined as follows:

SDI = (Strength of Stabilised sample - Strength of untreated sample) / Strength of untreated sample

Plots of SDI versus curing age of treated samples are shown in Figure 3. Figure 3 shows that for samples stabilised with any particular lime content, the values of SDI increases with increasing curing age. Figure 3 also shows that for samples cured at any particular age, SDI increases with the increase in lime content. Figure 3 clearly shows that the relative degree of strength gain resulted due to increasing curing age and lime content. As can be seen from Figure 3 that long-term curing has got significant effect on strength development. Rajbongshi (1997) and Hasan (2002) also reported increase in SDI with increasing curing time and lime content, as obtained in the present investigation.

Table 4 Unconfined Compressive Strength of Lime Stabilised Expansive Soil Samples

Cement Content (%)	Unconfined Compressive Strength, q_u (kN/m^2)				
	1 week	2 weeks	4 weeks	8 weeks	16 weeks
3	750	850	1100	1230	1350
6	1010	1120	1820	2450	3100
9	1100	1220	1930	2640	3450
12	1180	1350	2300	2840	3980
15	1440	1620	2650	3150	4600

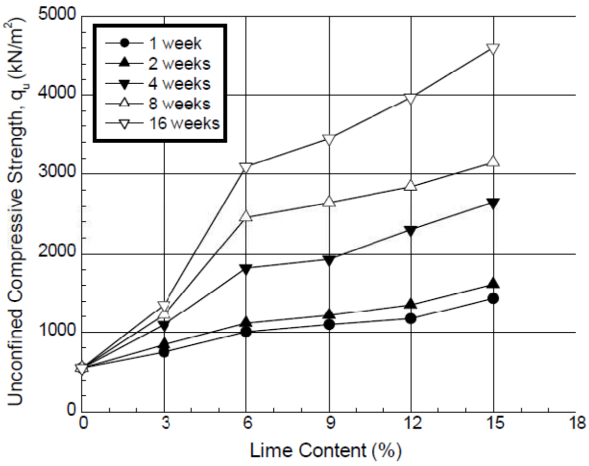

Figure 2 Effect of Lime Content on Unconfined Compressive strength

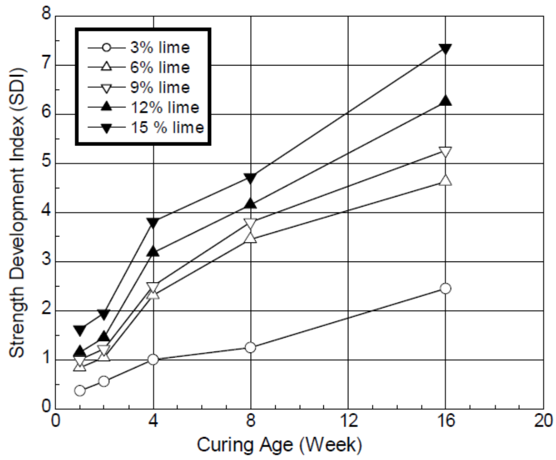

Figure 3 SDI versus Curing Curves for Lime Treated Expansive Soil

3.5 Effect of lime stabilisation on California Bearing Ratio (CBR)

A summary of the CBR test result for samples of the expansive soil is presented in Table 5. In order to investigate CBR-dry density relationship for untreated and stabilised samples, CBR tests were performed on samples compacted using three levels of compaction energies, e.g., low compaction energy (478 m-kN/m^3), medium compaction energy (1196 m-kN/m^3) and high compaction energy (2679 m-kN/m^3). It can be seen from Table 5 that compared with the untreated sample, CBR-values of the treated samples at all levels of compaction increased considerably.

The variation of CBR with lime content is shown in Figure 4, while Figure 5 presents the CBR versus compaction energy plots for the same samples. It can be seen from Figure 4 that at all levels of compaction, CBR increases markedly with increasing lime content while Figure 5 shows that at any particular lime content, CBR increases significantly with the increase in compaction energy. It has been found that compared with the untreated sample, CBR-values of treated samples increased by 4.75 to 8.75 times due to increase in lime content from 3% to 15%. Molla (1997) and Rajbongshi (1997) investigated the effect of lime on CBR-values of three regional soils and a coastal soil of Bangladesh, respectively. It was found that CBR value of stabilised samples increased with increasing lime content. Rajbongshi (1997) also reported that at any particular lime content, CBR increased significantly with the increase in compaction energy. TRB (1987) reported the effect of lime treatment on CBR-values for three plastic clays (liquid limit = 35 to 59, plasticity index = 15 to 30) and showed that for all the soils CBR increases markedly with increasing lime content.

Table 5 Summary of CBR Test Results of Untreated and Lime Treated Samples of Expansive Soil

Cement Content (%)	California Bearing Ratio (CBR)		
	Low Compaction Energy	Medium Compaction Energy	High Compaction Energy
0	4	5	8
3	26	29	38
6	30	32	41
9	35	37	46
12	41	50	60
15	52	58	70

Note: Low compaction energy = 478 m-kN/m^3
Medium compaction energy = 1196 m-kN/m^3
High compaction energy = 2679 m-kN/m^3

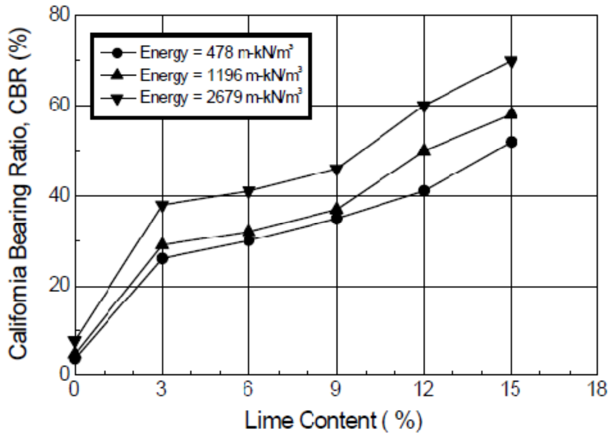

Figure 4 Effect of Lime Content on CBR

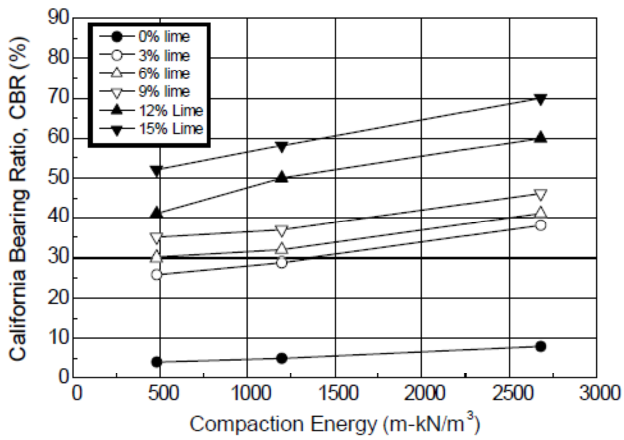

Figure 5 Effect of Lime Compaction energy on CBR

4. CONCLUSIONS

The major conclusions may be summarised as follows:

- Plasticity index, linear shrinkage, volumetric shrinkage and shrinkage ratio of samples of lime treated expansive soil decreased markedly due to stabilisation.
- Free swell, free swell index, swelling pressure, swelling potential and volume change of the stabilised samples decreased with increasing lime content. Influence of lime stabilisation on swelling properties of the expansive soil clearly demonstrates that lime can be considered a very effective additive in reducing the various swelling properties of an expansive soil.
- Values of γ_{dmax} reduced while the values of w_{opt} increased with increasing lime content.
- Unconfined compressive strength of the treated samples increased significantly, depending on the lime content and curing age. Strength development index increases with increasing curing age and

increasing lime content. Long-term curing has marked influence on the gain in strength.

- CBR-values of the treated samples increased considerably with increasing lime content. At any particular lime content, CBR increased significantly with the increase in compaction energy.

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THE STUDY ON TRIP GENERATION MODEL OF RESIDENTIAL BUILDING IN BANGKOK AREA

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ABSTRACT:

There has been blooming development in high density residential building in Bangkok and its surrounding area. Such residential buildings generate a lot of traffic volume additional to the existing demand on the road network. This study tries to estimate number of vehicle trip ends depending on weekday and weekend behavior. Multiple regression analysis with stepwise method was applied to develop the relationship between vehicle trip ends and each independent variable. In weekday behavior, there were four potential independent variables including number of residential unit (X2), total recreational area (X7), the number of route of directly passing public bus/van (X9), and number of entrance and exit point (X12) in each sample. In contrast, in weekend behavior, there were five potential independent variables which were X2, unit price (X4), X9, the distance between each sample and main road (X10), and X12. Better estimating trip generation is recommended for a reduction of the traffic impact surrounding the residential building. Consequently, more efficient and safer travelling of residents can be achieved.

KEY WORDS: Trip ends, Dependent Variable, Independent Variable

1. INTRODUCTION

Recently, high density residential building industry has been massively developed in Bangkok and its suburbs. Such buildings generate a number of subsequent trips which cause problematic transportation network especially at the bottle neck area such as an entrance and an exit. Thus, an estimation of the number of trips is very important for traffic management. The proposed trip generation on new residential buildings is meaningful when precised values are obtained. Trip generation analysis is a proper alternative to estimate number of trip ends, which can separate into two groups as weekday and weekend character. Therefore, transportation planners are able to use number of trip ends in order to express the mitigation measures, which compatible with real time traffic condition and their network.

2. SAMPLE SELECTION AND DATA COLLECTION

A selection of sample residential buildings in this study is based on following factors:

- A distance between the building and main street is less than 200 meters or within a walking distance. Residents can easily access to major public transport systems.
- A unit per building is more than 500 units

- Major purpose of building area utilization is for residence with less than 10 percents of commercial area.

Due to the budget limitation, the study randomly selected only seven representative residential buildings in Bangkok. The positions of the samples were roughly described (Table 1). The location of the samples is shown in Figure 1.

Table 1 Sample in the Study

No.	Roughly position	Units
1	Pradipat Soi 12	1,093
2	Ramindra Rd. (KM. 2)	2,839
3	Ramkhamhaeng 24 Rd.	636
4	Ramkhamhaeng Rd. (Soi 44)	827
5	Ramkhamhaeng 26 Rd.	1,089
6	Saereethai Rd.	1,650
7	Rattanathibath Rd.	576

Data collection was conducted for both weekday and weekend behavior as detailed below:

Y1 = 16 hours trip ends in passenger car unit (pcu.) on weekday behavior (8.00 a.m. to 12.00 p.m.)
Y2 = 16 hours trip ends in passenger car unit (pcu.) on weekend behavior (8.00 a.m. to 12.00 p.m.)
X1 = total building area in square meter
X2 = number of residential unit in each sample
X3 = average unit area in square meter (Determined using the ratio between number of room in each unit size multiply by its unit area and total of unit in the sample)
X4 = unit price in Baht. (The lowest price in each sample was used.)
X5 = parking area in square meter
X6 = total commercial area in each sample in square meter
X7 = total recreational area in each sample in square meter
X8 = the ratio between building area and parking area
X9 = the number of route of public bus/van which directly passing each sample.
X10 = the distance between each sample and main road in meter.
X11 = overall distance of service road in each sample in meter (only public zone)
X12 = number of entrance and exit point in each sample
X13 = the width of main service road in each sample in meter.

All obtained data of each sample were shown in Table 2.

Table 2 Data Collection in The Study

Variable/Sample No.	1	2	3	4	5	6	7
Y1 (pcu./16hr.)	1,584	2,942	854	1,794	1,346	1,865	820
Y2 (pcu./16hr.)	1,625	3,245	720	1,643	1,464	1,754	595
X1 (m2)	83,000	114,500	41,460	59,000	43,470	128,895	14,400
X2 (units)	1,093	2,837	636	827	1,089	1,650	576
X3 (m2)	60.00	44.75	75.00	55.00	45.25	57.91	30.00
X4 (Baht)	1,000,000	900,000	960,000	1,800,000	1,330,000	1,090,000	450,000
X5 (m2)	12,818	24,192	10,070	6,120	11,058	19,830	4,114
X6 (m2)	436	275	0	648	231	3,540	90
X7 (m2)	2,251	19,250	2,969	3,888	1,425	1,500	250
X8	6.48	4.73	4.12	9.64	3.93	6.50	3.50
X9 (no. of route)	12	22	0	40	0	9	7
X10 (m.)	0	0	115	0	0	0	0
X11 (m.)	130	300	47	141	437	200	70
X12 (point)	1	2	1	1	1	2	2
X13 (m.)	8	7	10	8	8	6	6

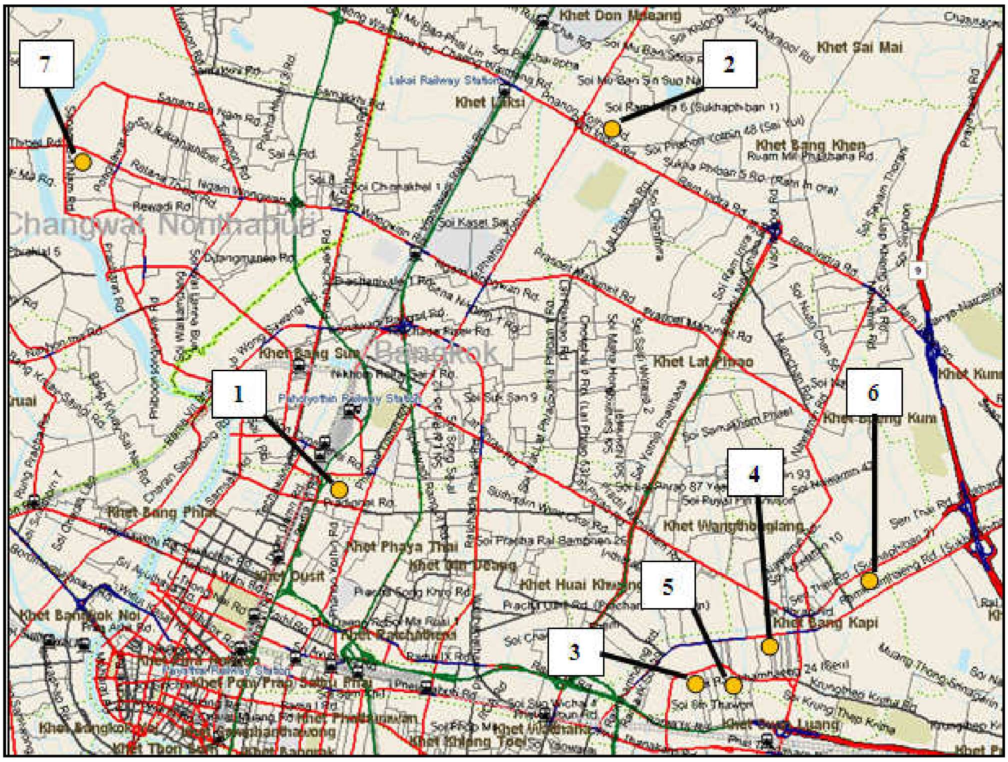

Figure 1 The Location of Selected Samples

3. IDENTIFYING POTENTIAL CONTRIBUTION FACTORS

The development of potential contribution factors was conducted before performing regression analysis. A potential contribution factor is any factor that could contribute significant generating trip to and from a residential building. The potential contribution factors serve as independent variables, while Y1 and Y2 serve as dependent variable in weekday and weekend behavior, respectively.

In weekday behavior, there were four potential independent variables which were X2, X7, X9, and X12, whereas, in weekend behavior, there were five potential independent variables including X2, X4, X9, X10, and X12.

4. TRIP GENERATION MODEL

Since weekdays and weekends have different trip generation rate patterns, two models were developed for each independent variable. One is for weekdays and the other is for weekends.

The multiple regression analysis with stepwise method produces two trip generation models as shown in equation 1 and 2.

$$Y1 = 492.706 + 0.983*(X2) + 19.036*(X9) - 188.842*(X12) - 0.020*(X7) \quad \text{(Eq.1)}$$

$$Y2 = 779.589 + 1.101*(X2) - 5.9*10^{-5}*(X4) + 12.501*(X9) - 441.521*(X12) - 2.272*(X10) \quad \text{(Eq.2)}$$

From equation 1, the independent variables including X2, X9, X12 and X7 were selected with coefficient of determinations (R^2) equal

0.862, 0.983, 0.993 and 1.000 respectively. Furthermore, equation 2, the independent variables including X2, X4, X9, X12 and X7 were selected with coefficient of determinations equal 0.892, 0.960, 0.976, 0.991 and 1.000 respectively. Results of stepwise model of weekday and weekend behavior were shown in Table3 and Table4, respectively.

Table 3 Results of Weekday Behavior[a]

	MODEL	R	R^2	EQUATIONS
Y1	1	.929	.862	Y1 = 542.427 + 0.851(X2)
	2	.991	.983	Y1 = 402.083 + 0.775(X2) + 18.251(X9)
	3	.996	.993	Y1 = 568.844 + 0.836(X2) + 17.379(X9) - 162.373(X12)
	4	1.000	1.000	Y1 = 492.706 + 0.983(X2) + 19.036(X9) - 188.842(X12) - 0.020(X7)

[a]See sample selection and data collection for abbreviations

Table 4 Results of Weekend Behavior[a]

	MODEL	R	R^2	EQUATIONS
Y2	1	.945	.892	Y2 = 284.203 + 1.040(X2)
	2	.980	.960	Y2 = 1.057(X2) + 0.001(X4) - 321.961
	3	.988	.976	Y2 = 1.009(X2) + 0.0003(X4) + 9.963(X9) - 177.863
	4	.996	.991	Y2 = 502.118 + 1.105(X2) + (1.29×10^{-5})(X4) + 14.094(X9) - 345.579(X12)
	5	1.000	1.000	Y2 = 779.589 + 1.101(X2) - (5.9×10^{-5})(X4) + 12.501(X9) - 441.521(X12) - 2.272(X10)

[a]See sample selection and data collection for abbreviations

In weekday behavior, X2 and X9 demonstrated their positive coefficients. The additional of one unit in sample may generate only one trip end (in pcu./16hr. unit) while the additional of one route of public bus/van directly passed the sample might generate approximately 19 trip ends. Therefore, the additional in route of public transport surrounding residential building is the most important factor affecting number of trip ends. The positive coefficient of X9 might not represent a rational signal. Typically, the more public transportation route resulted the less vehicle trip ends. Contrary, this study found that more public transportation route implied higher number of person trips on the road network. Thus, the higher person trips on the road might provide the positive effect to the vehicle trip ends of each sample.

On the other hand, X12 and X7 represented negative coefficients. X12 demonstrated that, as a consequent of only one point increased in entrance and exit, the number of trip ends might decline approximately 190 trips. The results suggesting that the additional entrance and exit point may reduce the security of the buildings. Due to anyone can easily access to the building, thus the resident-to-be may consider about lacking of privacy. Hence, X7 demonstrated no effect of the recreational area on the number of trip end since its coefficient was only 0.020.

In weekend behavior, X2 and X9 exhibited positive coefficients resembling the results of weekday. An addition of one unit in sample might generate only one trip end, whilst the addition of one route of public bus/van directly passed the sample might generate approximately 12-13 trip ends. Thus, these can be assumed that increased bus/van route provided stronger impact on the trip end of weekday.

Contrary, X4, X12, and X10 depicted negative coefficients. Only X12 showed similar result to the weekday behavior with greater impact. The only one entrance and exit increase, the number of trip ends were reduced 440 trips. The futures residents may strongly consider in relax lifestyle and privacy, so passing by strangers are undesired. When the additional unit price (X4) was 100,000 baht per unit, trip ends were reduced approximately six trips. The higher price of building unit implies the more coziness for the future residents. Coziness in the residence is important as the residents would love to be at home rather than going out for any purpose. At last, X10 was only one variable illustrated negative coefficient. One meter increased of the distance between residential buildings to main road affected the declined number of trip ends for approximately 2 trips. This is because everyone spent their time at home during weekends, thus less travels were generated.

5. CONCLUSIONS

The study developed trip generation model on residential building by using stepwise method. The models include weekday and weekend characteristics. The conclusions of the study are as follows:

- Number of unit (X2) and number of route of bus/van directly passing the sample (X9) are two main factors in order to give the positive value of trip ends in both weekday and weekend behavior.
- Number of entrance and exit point (X12) is the one factor that decrease the number of trip ends in both weekday and weekend behavior. X12 is the most important factor to decrease number of trip ends in the study
- Unit price (X4) and distance between sample and main road (X10) are two factors that decrease number of trip ends in weekend behavior only.

6. RECOMMENDATIONS

Due to budget limitation, the study randomly selected only seven samples in order to find out the trip generation model. To produce the trip generation model more precisely, the number of sample must be added

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ANALYSIS OF DRIVING CONDITIONS AND PREFERRED MEDIA ON DIVERSION

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ABSTRACT:

Studies on the distribution of traffic demands have been proceeding by providing traffic information for reducing greenhouse gases and reinforcing the road's competitiveness in the transport section, however, since it is preferentially required the extensive studies on the driver's behavior changing routes and its influence factors, this study has been developed a discriminant model for changing routes considering driving conditions including traffic conditions of roads and driver's preferences for information media. It is divided into three groups depending on driving conditions in group classification with the CART analysis, which is statistically meaningful. And the extent that driving conditions and preferred media affect a route change is examined through a discriminant analysis, and it is developed a discriminant model equation to predict a route change. As a result of building the discriminant model equation, it is shown that driving conditions affect a route change much more, the entire discriminant hit ratio is derived as 64.2%, and this discriminant equation shows high discriminant ability more than a certain degree.

KEY WORDS: Diversion, Driving conditions, Preferred media, Discrimint model, CART analysis

1. INTRODUCTION

1.1 Backgrounds and Purposes

Interests in the greenhouse gas reduction and the environmental friendly green growth are increasing as the global warming problems are aggravated. It is more and more interested in how to efficiently use the existing roads in order to reduce greenhouse gases and reinforce road's competitiveness in the transport section, and studies on the distribution of traffic demands by providing traffic information are in progress as a practical method for them. However, driver's decision making to change routes is not a problem whether or not the information is simply provided, but may be varied depending on driving conditions or preferred media when it is provided. In other words, driver's responses for route changes may be varied depending on the driving conditions such as weather situations and traveling distances etc. and the individual driver's preferred property for media to provide information even though the same information is provided. In particular, even though a result of driver's decision making to change routes is represented as a simple result whether to drive the existing route or change the route, the process to actually determine it is carried out through a diverse, complex and step by-step process such as examining various environmental conditions, considering the worst situation, and ranking priority in accordance with the preferred property for each person. Therefore, it is needed a study on such a decision making process to change routes, if sufficient studies are not accomplished, drivers may respond differently each other for the same traffic information, furthermore, it is important that the fact they may respond inconsistently from operator's intention is not overlooked.

1.2 Scope of Work and Methods

In terms of the importance for such a decision making to change routes, this study would develop a discriminant model to change routes considering the driving conditions including traffic situations on the road and the driver's preferred property for information media. For an analysis, it is investigated whether or not to change routes followed by traffic situation on the road and preferred media by carrying out a survey for drivers using expressways. To efficiently collect and classify data to be examined, the tree analysis method for decision making is used, and they are re-classified according to driving conditions representing statistically meaningful collective differences for whether or not to change routes. In addition, the extent that driving conditions and preferred media affect a route change is examined through a discriminant analysis, and a discriminant model equation is developed to predict a route change.

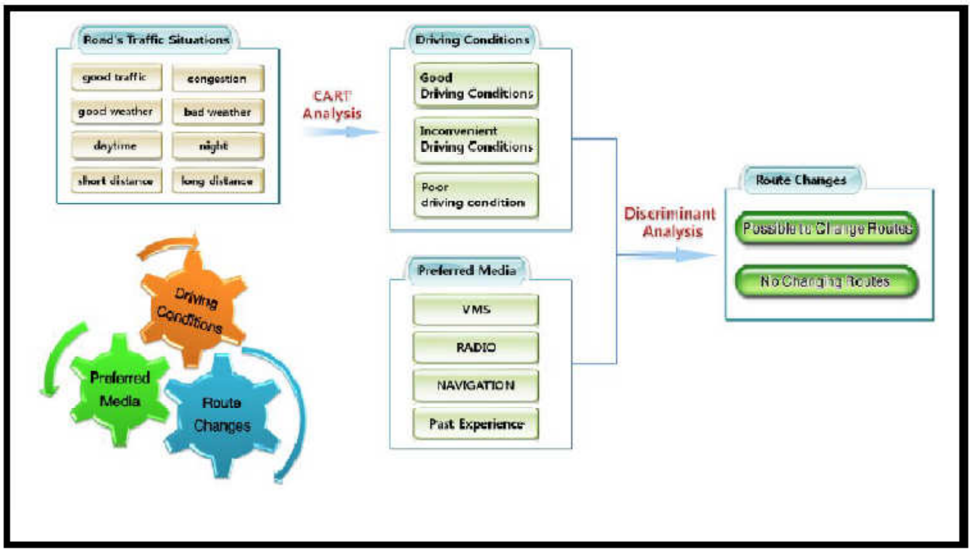

Figure 1 Research Method

2. BACKGROUNDS

2.1 Literature Review

Studies related to providing the traffic information are divided into information provision, strategies to provide information, evaluations of information provision effects, system configurations to provide information, and recently studies on the driver's response behavior followed by providing information are primarily carried out.

Khattak (1993) presented a result that driver's decision making was affected in the order of their past experiences (63%), direct observation (21%), and traffic information (16%) in his study on the driver's selection for actions, and H. R. Kim et al. (2004) identified that there were traveler's property, travelling property, route property, information content, their past experiences etc. for factors affecting driver's decision making to change routes, and it was affected by inherent property of media, form of information, and delivered content etc. when drivers made a decision to change routes after receiving information from media.

Y. H. Choi et al. (2007) identified in their study on the information media affecting driver's decision making to change routes that drivers depended on traffic information more than their own experiences as the congestion was more serious, and I. P. Kim (2008) found a primary factor deciding a detour through a preference analysis, and identified that a strategy was needed to provide each traffic information for each traffic situation that was a core factor of route changes. Meanwhile, Khattak et al. (2008) presented that media such as Internet, radio broadcasting, VMS, navigation etc. among information provision media affecting driver's behavior had affected the decision making of destinations, travelling means, route selections etc. In particular, when comparing the existing study's result of Khattak (1993, 2008), it could be found that individual driver's experiences were regarded as important in the 1990s, but preference for information media has been increasing more than personal experiences since the high reliable traffic information is timely provided via various media due to development of information communication technologies from the 2000s. Nevertheless, a remarkable fact is that even now, there are drivers who select routes using the past experiences in spite of the information age, and it is not irrelevant to user experiences (Ux) becoming an issue recently. The Ux means overall experiences that users feel and think as they directly or indirectly use any system, product, and service. It is a valuable experience that users could know by participating, using, observing, and interacting in every whole perceivable aspect as well as satisfaction on functions or procedures simply. Referring to the results of Khattak (1993) and Y. H. Choi et al. (2007), it is seemed that the past experiences such as user's direct driving experiences and indirect travelling experiences by persons around them affect a route selection primarily in the traffic aspect.

Y. H. Choi et al. (2009) identified that there was a meaningful relationship between driver's detour behaviors and road's traffic situations through a correlation analysis between the bypass rate investigated actually and the road's traffic situation (traffic volumes on the main line, travelling speed, and travelling time), and derived a regression equation for the expressway's bypass rate by using it. In addition, Y. H. Choi et al. (2010) analyzed how the use pattern of information media was varied according to traffic situations, and re-classified into passive use media, active use media, the past experiences based on the property for each medium to analyze variation of the bypass use rate for each information medium followed by each traffic situation. They identified that while the use rate of a passive use medium was decreased as the traffic situation became worse; the bypass rate using an active use medium and the past experiences was increased. S. N. Son (2010) identified that a will to change routes and driver's perceived behavior control for a route change most significantly affected a decision whether or not to change routes.

2.2 Problem statement

In spite of many studies related to traffic information like this, studies on the driver's decision making to change routes are insufficient. In particular, since it has been identified that driver's route changes depend on traffic situations and the traffic information media used for changing routes become different according to traffic situations, an analysis on the relationship between traffic situations, preferred media, and route changes is needed by integrating them. This study is a succeeding study of Y. H. Choi et al. (2009, 2010), which would comprehensively analyze driver's property to change routes considered fragmentarily such as traffic situations and route changes, traffic situations and properties to use traffic information media etc. according to the property of preferred media under driving conditions such as travelling distances including road's traffic situations and weather conditions etc.

3. ANALYSIS METHODS

This study performed an analysis with the following process.

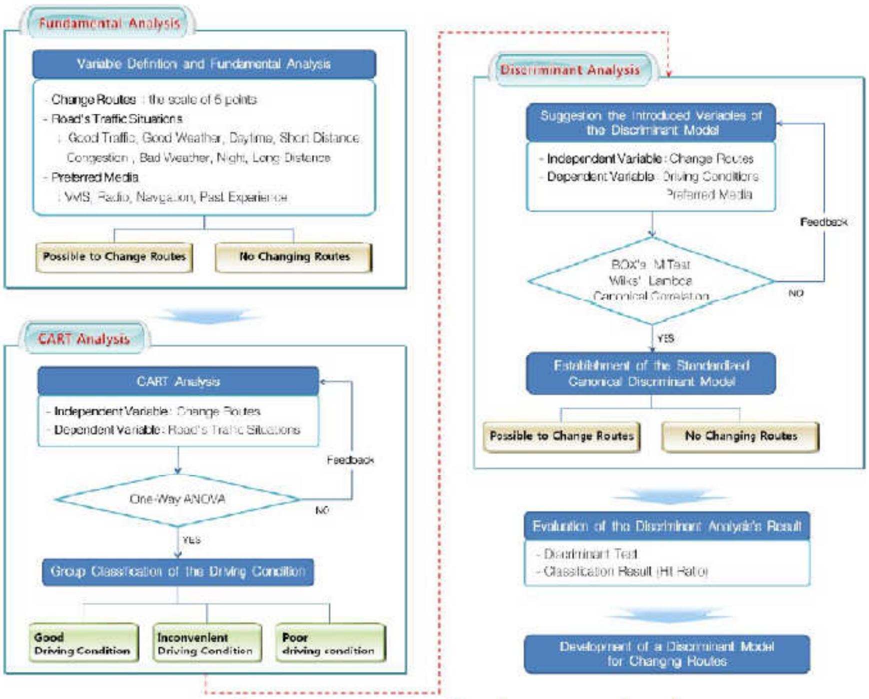

Figure 2 Analysis Method

First, it is performed whether or not to change routes for variables to be used in this study, definitions of the classification for road's traffic conditions and preferred media, and a fundamental statistical analysis.

Second, for the appropriate classification of road's traffic situations which is a core part of this study, the CART analysis method is used to investigate road's traffic situations representing statistical meaningful collective differences for route changes.

Third, a statistical verification is performed for the group classification of driving conditions by performing the one-way ANOVA for driving conditions classified by the CART analysis.

Fourth, a discriminant model is developed for driving conditions and preferred media affecting the route changes.

Fifth, the discriminant hit ratio is analyzed for the developed discriminant analysis model.

4. DATA ANALYSIS

4.1 Fundamental Analysis

This study performed a survey for total 500 persons using metropolitan expressways. Among total 500 of respondents, males occupied 60% as 286, and females occupied 40% as 214.

In the result of responses having multiple choices asking whether or not to change routes for 8 road's traffic situations and 4 preferred media, the analysis was performed by using 6,462 data that the preference for each medium was responded above 8 points. It was because drivers having a preference of 'above favor' for the preferred media in each road's traffic situation were considered to represent well the relationship between the road's traffic situations and the preferred media and the relationship between the preferred media and whether or not to change routes as drivers affected by the preferred media when changing routes.

The questions about whether or not to change routes would be responded whether or not to change routes for each question on the scale of 5 points from 'no changing routes at all' to 'always possible to change routes.' The analysis was performed by grouping the point between 1 to 3 into 'no changing routes' and the point between 4 to 5 into 'possible to change routes' among responses for questions of the 5-point scale.

As a result of the frequency analysis for whether or not to change routes as a target variable, it was analyzed that the responses of the possible to change routes occupied 46.7%, and the responses of the no changing routes occupied 53.3%. Since the respondents responding as 'average (3 points)' who occupied 25% of the entire respondents are classified into the no changing routes, it is considered that a little conservative response result was come out.

Since the major purpose of decisions for whether or not to change routes that would be presented in this study is also to find the possibility to change routes by drivers who use the corresponding road when variables for preferred media of the drivers are entered for each road's traffic situation, this study would be approached in a little conservative aspect by using the result of above survey.

Table 1 Analyzing The Fundamental Data for Whether or Not to Change Routes

Type	No Changing Routes			Possible to Change Routes	
	1 (No Changing Route at All)	2	3	4	5 (Always Possible to Change Route)
Frequency	828	999	1616	1806	1213
Percent (%)	12.8	15.5	25.0	27.9	18.8
	53.3%			46.7%	

4.2 Group Classification of the Road's Traffic Situations with the CART Analysis

The CART analysis method was used to classify the road's traffic situations considered as a major factor affecting driver's route changes, and the groups of road's traffic situations representing the statistical meaningful collective differences for whether or not to change routes were reclassified depending on driving conditions. In addition, a statistical verification between groups re-classified depending on driving conditions was performed with the one-way ANOVA for the groups classified by the CART analysis.
First, as a result of the CART analysis based on whether or not to bypass, they were classified into 'node 1' (smooth traffic, good weather, day, short distance) and 'node 2' (poor driving conditions, bad weather, night, long distance) as the Figure 3. The result of the statistical significance test for the classification between groups represented the p-value for the t-test as 0.000, so it could be said that the difference between two groups was significant for 95% level of the reliability. The result of the t-test analysis for the 'node 3' (poor driving conditions) and the 'node 4' (bad weather, night, long distance) for the node 2 also represented the p value for the t-test as 0.000, so it could be said that the difference between two groups was significant for 95% level of the reliability.

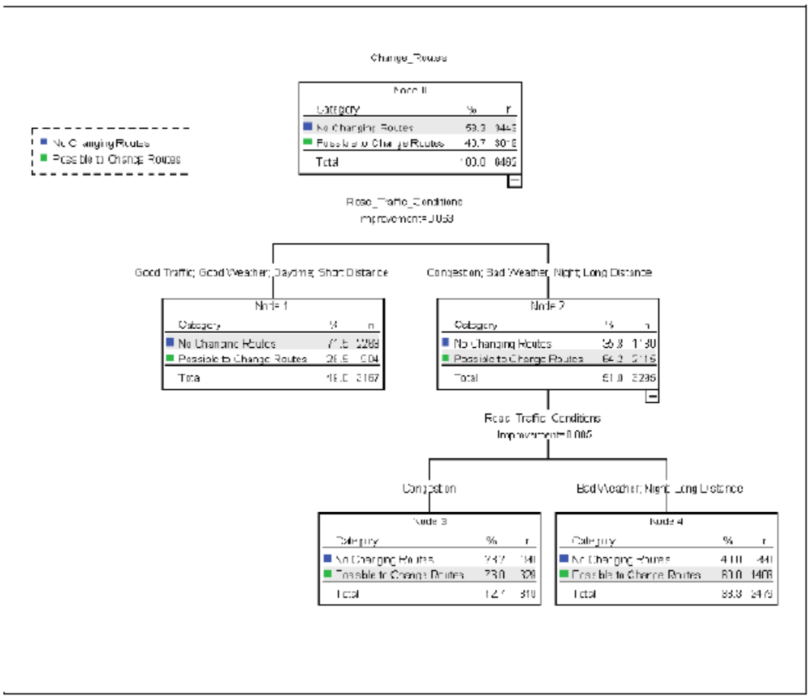

Figure 3 Group Classification of the Road’s Traffic Situation with The CART Analysis

Table 2 Result of the Statistical Significance Test
for the Classification between Groups

Parent Node	Child Node	N	Mean	Std. Deviation	Levene’s Test of Equality of Error Variances		t-test	
					F	p-value	t-value	p-value
Node 0	Node 1	3167	1.71	0.452	155.615	0.000	30.731	0.000
	Node2	3295	1.36	0.480				
Node 2	Node 3	819	1.23	0.423	468.556	0.000	-8.764	0.000
	Node 4	2476	1.40	0.490				

Based on the groups classified by the CART analysis, the road's traffic situations were reclassified into three groups of driving conditions to perform the post analysis.

- Group A (good driving conditions): good traffic, good weather, daytime, short distance
- Group B (inconvenient driving conditions): bad weather, night, long distance
- Group C (poor driving conditions): congestion

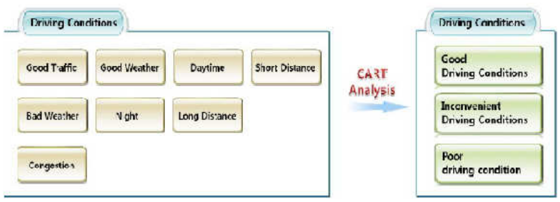

Figure 4 Group Classification of the Driving Condition

The result of the one-way ANOVA and the post analysis to verify the difference between three groups classified by the driving condition represented the p-value as 0.000, and the difference between groups depending on the driving condition was the same as the CART analysis, so it had been proved that the group classification with the CART analysis was statistically appropriate.

4.3 Development of a Discriminar Model for Changing Routes

4.3.1 Outline

The "1" of nominal scale was assigned to drivers changing routes, and the "0" of nominal scale was assigned to drivers not changing routes depending on whether or not to change routes for drivers using their own preferred media for each driving condition. The analysis with the stepwise method was performed to understand variables affecting the route changes significantly, and the Wilks' method was used as the criteria to select variables.

4.3.2 Consideration of the Discriminant Analysis's Result

(1) Analysis of Covariance Matrix

The discriminant analysis is established under the assumption. It has a multi-variate normal distribution and the covariance of each group is identical. The homogeneity test result of the covariance matrix analyzed through the Box test for groups represented the significance probability of the Box test as 0.139, and it was selected the null hypothesis that their covariance are identical, so it could be defined that the covariance matrixes of two groups are identical.

Table 3 Result of the One-way ANOVA for Each Driving Condition

Driving Condition	Mean	Std. Deviation	Levene's Test of Equality of Error Variance		One-Way ANOVA		Post Hoc Tests
			F-value	Sig.	F-value	Sig.	
Good Driving Conditions (a)	2.71	1.167	139.251	0.000	2337.863	0.000	$a \neq b \neq c$ (Scheffe Test)
Inconvenient Driving Conditions (b)	3.65	1.168					
Poor Driving Condition (c)	4.12	0.910					

Table 4 Summarizing the Introduced Variables of the Discriminant Model

Dependent Variable	Independent Variable	
Whether or Not to Change Routes	Driving Conditions (a)	Good Driving Conditions (a1)
		Inconvenient Driving Conditions (a2)
		Poor Driving Conditions (a3)
	Preferred Media (b)	VMS (b1)
		Radio (b2)
		Navigation (b3)
		Past Experience (b4)

Table 5 Box's Test Results

Box's M	122.830
F Approx.	1.831
df 1	3
df 2	1.64710
Sig.	0.139

(2) Significance Test of the Discriminant Model

As a result of the discriminant analysis, a discriminant function was derived, and it was analyzed that both the driving conditions and the preferred media had the statistically significant effects.

Even though this analysis represented the canonical correlation coefficient as 0.310, the Eigenvalue as 0.107, the Wilks' lambda value as 0.904, which the discriminant were not very high, but the significant probability of the chi square test was 0.000 < =0.05, which represented that it was statistically significant. Examining the Wilks' lambda for road's traffic situations and preferred media and the value converting it into the F statistics also in the analysis on the discriminant factors, since the significant probability of the F statistics was less than 0.05, it could be said that the average difference of the discriminant score between groups was significant.

The size of coefficient's absolute value in a discriminant equation represents the relative importance between variables, and it could be found that the driving conditions have more effects than the preferred media when it was examined the result analyzing the coefficients of the standardized canonical discriminant function for these discriminant factors.

Table 6 Summary of Canonical Discriminant Functions

Eigen Values	Eigen Values	% of Variance	Cumulative %	Canonical Correlation
	0.107	100.0	100.0	0.310
Wilks' Lambda	Wilks' Lambda	Chi Square	df	Sig.
	0.904	653.97	2	0.000
Test of Equality of Group Means	Type	Wilks'	F	Sig.
	Driving Conditions	0.909	649.788	0.000
	Preferred Media	0.999	8.727	0.003

Table 7 Standardized Canonical Discriminant Function Coefficients

Variable	Function	
	Standardized Canonical Discriminant Function Coefficients	Canonical Discriminant Function Coefficients
Driving Conditions	1.002	1.132
Preferred Media	-0.239	-0.216
(Constant)	-	-1.635

(3) Establishment of the Standardized Canonical Discriminant Model

The discriminant model equation was defined for whether or not to change routes by using the coefficients of the canonical discriminant function (non-standardized canonical discriminant function).

$$Z = -1.635 + 1.132 \times XF_i - 0.216 \times XMj \quad \text{(Eq.1)}$$

Here, Z: Whether or not to change routes

XF_i: Driving Condition
(i: 1 = Good Driving Conditions,
2 = Inconvenient Driving Conditions,
3 = Poor Driving Conditions)
XM_j: Preferred Media
(j: 1 = VMS, 2 = Radio,
3 = Navigation, 4 = Past Experience)

If the discriminant score found by the above discriminant equation is larger than the classification standard, it is classified into the group 1 (possible to change routes), or if it is smaller, it is classified into the group 2 (no changing routes.) The average discriminant score of the group 1 is 0.349, and the average discriminant score of the group 2 is -0.306, which the standard of classification is calculated as the average of the cutting point of these two groups.

$$Centroid = \frac{\sum_{i=1}^{m} n_i C_i}{\sum_{i=1}^{m} n_i} = \frac{n_1 C_1 + n_2 C_2}{n_1 + n_2}$$

(Eq.2)

Here; n_i: Sample size of group i
C_i: Centroid of group i

The standard of classification is derived as the 0 of cutting point according to the above equation. Therefore, the case of having a larger value than the 0 by the discriminant model is classified into the group 1 (possible to change routes,) or if the value is less than the 0, it is classified into the group 2 (no changing routes.)

Table 8 Function at group Centroid

Changing Route	Function 1	
	Centroid	Cutting Point
Possible to Change Routes	0.349	0.0
No Changing Routes	-0.306	

(4) Establishment of the Discriminant Model for Changing Routes

Using a result of the Fisher's linear discriminant analysis, the linear discriminant function for each group could be derived to determine a bypass.

Z (Possible to Change Routes) $= -5.365 + 2.543 \times XF_i + 1.638 \times XM_j$
Z (No Changing Routes) $= -4.282 + 1.803 \times XF_i + 1.779 \times XM_j$

(Eq.3)

Here, Z: Whether or not to change routes
XF_i: Driving Condition
(i: 1 = Good Driving Conditions,
2 = Inconvenient Driving Conditions,
3 = Poor Driving Conditions)
XM_j: Preferred Media
(j: 1 = VMS, 2 = Radio,
3 = Navigation, 4 = Past Experience)

Table 9 Classification Function Coefficients

Type	Whether or Not to Change Routes	
	Possible to Change Routes	No Changing Routes
Driving Conditions	2.543	1.803
Preferred Media	1.638	1.779
(Constant)	-5.365	-4.282

(5) Discriminant Test

Examining the result performing the discriminant for each group based on data used in the analysis in order to verify the discriminative strength of the discriminant model developed as the result of the discriminant analysis, total discriminant hit ration is derived as 64.2%. The discriminant hit ratio of the group 1 (changing routes) is represented as 60.1%, and the group 2 (no changing routes) is represented as 67.9%. It could be said that the discriminant by this discriminant equation shows higher discriminative strength than a certain level.

Table 10 Classification Result

Type		Predicted Group Membership: Possible to Change Routes	Predicted Group Membership: No Changing Routes	Total
Count	Possible to Change Routes	1813	1206	3019
	No Changing Routes	1105	2338	3443
Ratio	Possible to Change Routes	60.1	39.9	100.0
	No Changing Routes	32.1	67.9	100.0

5. CONCLUSIONS
5.1 Conclusions

More efficient and effective strategies to provide the traffic information and control the traffic should be established in order to relieve the confusion and solve the congestion, since the study on the driver's behavior to change routes and the factors affecting it is preliminarily needed for the purpose, this study would develop the discriminant model to change routes considering the driving conditions including the road's traffic situations and the driver's preferred property for the information media.

To find the relationship between the preferred media for traffic information and the driving conditions affecting the driver's decision to change routes, the CART analysis method is used to re-classify the group of road's traffic situations, which represents the statistically significant collective differences for whether or not to change routes, depending on the driving conditions. In addition, it was examined the extent that the driving conditions and preferred media affect the route change through the discriminant analysis and it was developed the discriminant model equation to predict whether or not to change routes. Examining the result of building the discriminant model equation, it was represented that the driving conditions affected the route change much more, and it could be said that this discriminant equation showed higher discriminative strength than a certain level because the entire discriminant hit ratio was derived as 64.2%.

① The Standardized Canonical Discriminant Model
$Z = -1.635 + 1.132 \times XF_i - 0.216 \times XM_j$

② The Discriminant Model for Changing Routes
Z (Possible to Change Routes) $= -5.365 + 2.543 \times XF_i + 1.638 \times XM_j$
Z (No Changing Routes) $= -4.282 + 1.803 \times XF_i + 1.779 \times XM_j$

Here, Z: Whether or Not to Change Routes
XF_i: Driving Conditions (i : 1=Good Driving Conditions, 2=Inconvenient Driving Conditions, 3=Poor Driving Conditions)
XM_j: Preferred Media (j : 1=VMS, 2=Radio, 3=Navigation, 4=Past Experience)

5.2 Recommendations

Even though this study divided the road's traffic situations into three groups of driving conditions to analyze them, it is needed to analyze for whether or not to change routes by classifying the road's traffic situations in more detail in order to establish more efficient and effective strategies to provide the traffic information and control the traffic.

In addition, even though this study carried out the survey for the preferred media and whether or not to change routes depending on the road's traffic situations, an additional study is required for whether the driver's responses and whether or not to change routes acquired range routes survey are also identical to actual sites. It should be found the various causes of changing routes, the priority of each cause, and the correlation between them for drivers changing routes actually, it should check whether the results of the survey is identical to driver's actual behavior through the factor analysis, and it should perform the correlation analysis.

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AN ENHANCED COMBINED TRIP DISTRIBUTION AND ASSIGNMENT PROBLEM

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ABSTRACT:

This paper proposes a mathematical formulation and a solution method to the enhanced combined trip distribution and traffic assignment. The trip distribution is a doubly-constrained gravity model. The traffic assignment is the paired-combinatorial-logit stochastic user equilibrium accounting for effects of congestion, stochastic perception error and path similarity. This is an enhancement to existing multinomial-logit (MNL)-based model. The proposed solution method is a disaggregate simplicial decomposition algorithm. I find that the relationship of O-D flow difference and dispersion factor is unclear, whereas link flow patterns from the two models are more identical at higher dispersion factors. The enhanced model assigns less flow to a path with higher average similarity index and higher path cost than MNL model. The enhanced model generally assigns less flow to links with more paths passing through than MNL model. The relationship between O-D flow allocation and the average similarity indices for O-D pairs is not obvious.

Key Words: stochastic user equilibrium, gravity model, combined travel demand model

1. INTRODUCTION

The combined distribution and assignment (CDA) problem is an instance of combined travel demand models. CDA simultaneously determines the distribution of trips between origins and destinations in a transportation network and the assignment of trips to routes in each origin-destination pair. The trip distribution is mostly assumed to be a gravity model with a negative exponential deterrence function. The static trip assignment is either user equilibrium model (UE) or stochastic user equilibrium model (SUE). UE assumes that drivers have complete and accurate information on the state of the network when they make their route choices, and drivers select optimal routes to benefit themselves the most. SUE assumes that trip assignment follows a probabilistic route choice model. The multinomial logit-based SUE model (MNL-SUE) is widely adopted in the literature. Evans (1976) formulated the CDA problem that integrates the gravity-model trip distribution and user-equilibrium trip assignment (CDA-UE). Erlander (1990) formulated the CDA that integrates the gravity-model trip distribution and multinomial-logit stochastic-user-equilibrium assignment (CDA-MNL-SUE). Lundgren and Patriksson (1998) outlined the solution algorithms for CDA-UE and CDA-MNL-SUE.

With the property of independence of irrelevant alternatives (IIA) in the MNL model, the MNL-SUE has an infamous deficiency in the incapability to account for similarities between different routes. Although the multinomial probit-based SUE model by Daganzo and Sheffi (1977) can account for similarity between different routes, it is not attractive due to the lack of closed form of probability function. Over the past years, researchers adopted other discrete choice model structures to SUE in order to capture the similarity between routes on the perceptions and decisions of drivers while keeping the analytical tractability of the logit choice probability function. The SUE models based on the modifications of MNL are C-logit model and path-size logit model. The SUE models based on the generalized extreme value theory are paired combinatorial logit model (Prashker and Bekhor, 2000), cross-nested logit model, logit kernel model, link-nested logit model, and generalized nested logit model. Chen et al. (2003) pointed out that among these extended logit models, the paired combinatorial logit model (PCL) is considered the most suitable for adaptation to the route choice problem due to two features that can be employed to address the IIA property in the MNL model.

In this paper, I propose a combined gravity-model distribution and pairedcombinatorial -logit stochastic-user-equilibrium assignment formulation (CDA-PCL-SUE) and develop a disaggregate simplicial decomposition algorithm. The trip distribution model is doubly constrained such that both the total flow generated at each origin node and the total flow attracted to each destination node are fixed and known.

2. EQUIVALENT MATHEMATICAL FORMULATION

Denote by CDA-PCL-SUE the proposed combined distribution-assignment (CDA) equivalent mathematical model. The underlying route choice in CDA-PCL-SUE is a hierarchical route choice model that decomposes the choice probability into two levels. The upper level computes the marginal probabilities $P(kj)$ of choosing an unordered route pair k and j, based on the similarity index and the systematic utility.
The lower level is a binary logit model that computes the conditional probabilities of choosing a route given the chosen route pair: $P(k|kj)$ and $P(j|kj)$. The underlying trip distribution in CDA-PCL-SUE is a doubly constrained model that requires the O-D flows out of an origin node and into a destination node to be equal to the known origin demands and destination demands, respectively.

The definitions of sets, parameters, decision variables and mathematical formulation are given below, followed by the first-order conditions that are shown to be identical to the PCL-SUE equations and gravity-model based trip distribution equations.

Set

K_{rs} = set of routes between origin r and destination s

L_{rs} = set of unordered route pairs between origin r and destination s

R = set of origins

S = set of destinations

RS = set of origin-destination (O-D) pairs

A = set of arcs

Parameters

O_r = total trips originated from origin *r*

D_s = total trips destined to destination *s*

θ = dispersion coefficient

β_{kj}^{rs} = measure of dissimilarity index between routes *k* and *j* connecting O-D *r-s* ($\beta_{kj}^{rs} = 1 - \sigma_{kj}^{rs}$)

σ_{kj}^{rs} = measure of similarity index between routes *k* and *j* connecting O-D *r-s*

δ_{ka}^{rs} =1 if arc *a* is on route *k* connecting origin *r* to destination *s*; 0 otherwise

Decision Variables

x_a = flow on link *a*

t_a = travel time on link *a*

q_{rs} = demand between origin *r* and destination *s*

$f_{k(kj)}^{rs}$ = flow on route *k* of route pair *kj* between origin *r* and destination *s*

Mathematical Formulation

$$\min z = z_1 + z_2 + z_3 \quad \text{(Eq.1.1)}$$

$$z_1 = \sum_{a\in A} \int_0^{x_a} t_a(w)dw \quad \text{(Eq.1.1a)}$$

$$z_2 = \frac{1}{\theta} \sum_{r\in R} \sum_{s\in S} \sum_{k\in K_{rs}} \sum_{\substack{j\in K_{rs} \\ j\neq k}} \beta_{kj}^{rs} f_{k(kj)}^{rs} \ln \frac{f_{k(kj)}^{rs}}{\beta_{kj}^{rs}} \quad \text{(Eq.1.1b)}$$

$$z_3 = \frac{1}{\theta} \sum_{r\in R} \sum_{s\in S} \sum_{k=1}^{|K_{rs}|-1} \sum_{j=k+1}^{|K_{rs}|} (1-\beta_{kj}^{rs})(f_{k(kj)}^{rs} + f_{j(kj)}^{rs}) \ln \frac{f_{k(kj)}^{rs} + f_{j(kj)}^{rs}}{\beta_{kj}^{rs}} \quad \text{(Eq.1.1c)}$$

Subject to

$$\sum_{k\in K_{rs}} \sum_{\substack{j\in K_{rs} \\ j\neq k}} f_{k(kj)}^{rs} = q_{rs} \quad \forall r \in R, s \in S \quad \text{(Eq.1.2)}$$

$$\sum_{s\in S} q_{rs} = O_r \quad \forall r \in R \quad \text{(Eq.1.3)}$$

$$\sum_{r\in R} q_{rs} = D_s \quad \forall s \in S \quad \text{(Eq.1.4)}$$

$$f_{k(kj)}^{rs} \geq 0 \quad \forall k \in K_{rs}, kj \in L_{rs}, r \in R, s \in S \quad \text{(Eq. 1.5)}$$

$$x_a = \sum_{r\in R} \sum_{s\in S} \sum_{k\in K_{rs}} \sum_{\substack{j\in K_{rs} \\ j\neq k}} \delta_{ak}^{rs} \cdot f_{k(kj)}^{rs} \quad \forall a \in A \quad \text{(Eq. 1.6)}$$

The objective function (Eq.1.1) is composed of three components, similar to the objective of the PCL-SUE model. Eq.1.1a accounts for the congestion effects. Eq.1.1b and 1.1c are two entropy terms that represent the marginal and conditional probabilities in a hierarchical route choice model. Dissimilarity indices are incorporated into the objective function (Eq.1.1b and Eq.1.1c), allowing the model to capture the similarity effect and stochastic perception error effect in addition to the congestion effect (Eq.1.1a). Eq.1.2 enforces the summation of all path flows connecting an O-D pair to be equal to the O-D flows (q_{rs}) of this O-D pair. Eq.1.3 and Eq.1.4 are the O-D flow balance constraints for the origin nodes and destination nodes, respectively. Eq.1.5 are the non-negativity constraints for all path flow variables. Eq.1.6 determines the link flow variable from the summation of all path flows passing through this link. It is easy to show that the optimality conditions of the proposed formulation equal to the PCL formula in Eq.5.7-5.8 and the gravity-model based trip distribution equation:

$$q_{rs} = A_r B_s O_r D_s g(c^{rs}) \quad \forall r \in R, \ \forall s \in S \quad \text{(Eq.2)}$$

Where

$$A_r = \frac{\exp(\theta u_r - 1)}{O_r}; \ B_s = \frac{\exp(\theta v_s)}{D_s};$$

c^{rs} = vector of route travel times of O-D pair *r,s*;

$g(c^{rs}) =$

$$\sum_{k\in K_{rs}}\sum_{\substack{j\in K_{rs}\\ j\neq k}} \frac{\beta_{kj}^{rs}\cdot \exp\left(-\frac{\theta c_k^{rs}}{\beta_{kj}^{rs}}\right)}{\left(\exp\left(-\frac{\theta c_k^{rs}}{\beta_{kj}^{rs}}\right)+\exp\left(-\frac{\theta c_j^{rs}}{\beta_{kj}^{rs}}\right)\right)^{1-\beta_{kj}^{rs}}}$$

3. DISAGGREGATE SIMPLICIAL DECOMPOSITION ALGORITHM

The proposed algorithm for CDA-PCL-SUE is based on the disaggregate simplicial decomposition algorithm by Lundgren and Patriksson (1998) and Larsson and Patriksson (1992). The proposed algorithm alternates between two phases. In phase I (the restricted master problem), given known subsets of routes between O-D pairs $\hat{K}_{rs} \subseteq K_{rs}$ $\forall r \in R, s \in S$, of the total sets of routes in the network, the corresponding restriction of CDA-PCL-SUE (denoted by CDA-PCL-SUE-R) is solved approximately using a partial linearization descent algorithm, which is a descent algorithm for continuous optimization problems. In phase 2 (the column generation problem), at the approximate solution to CDA-PCL-SUE-R, the subsets $\hat{K}_{rs}$ are augmented by the generation of new routes, through the solution of a set of shortest path problems, given appropriately chosen link costs.

3.1. Phase I: Restricted Master Problem

The problem CDA-PCL-SUE-R is solved by a partial linearization descent algorithm (Patriksson, 1993). The projection of CDA-PCL-SUE-R onto the set of feasible route flows is employed. Given a feasible route flow vector $f^n = \{f_{k(kj)}^{rs^n}\}$ at some iteration *n*, an approximation of CDA-PCL-SUE-R is roughly solved in order to define an auxiliary feasible solution and a search direction. The approximate problem is constructed by linearizing the first term (z_1) of the objective function of CDA-PCL-SUE-R. The effect of this linearization is that the link costs are fixed at their levels given the current flow f^n; i.e. $\frac{\partial z_1(f^n)}{\partial f_{k(kj)}^{rs}} = c_k^{rs^n}$. The corresponding route costs are calculated as: $c_k^{rs^n} = \sum_{a\in A}\delta_{ka}^{rs}\cdot t_a(x_a^n)$ $\forall k \in \hat{K}_{rs}, r\in R, s\in S$, where x_a^n is the flow on arc *a* corresponding to the route flow f^n. The partially linearized problem (denoted by CDA-PCL-SUE-R-PL) becomes:

<u>Formulation of CDA-PCL-SUE-R-PL</u>

(the solution is denoted by $\underline{f}_{k(kj)}^{rs}$)

$$\min \tilde{z} = \tilde{z}_1 + \tilde{z}_2 + \tilde{z}_3 \quad \text{(Eq.3.1)}$$

$$\tilde{z}_1 = \sum_{r\in R}\sum_{s\in S}\sum_{k\in\hat{K}_{rs}}\sum_{\substack{j\in\hat{K}_{rs}\\ j\neq k}} c_k^{rs^n}\cdot f_{k(kj)}^{rs} \quad \text{(Eq.3.1a)}$$

$$\tilde{z}_2 = \frac{1}{\theta}\sum_{r\in R}\sum_{s\in S}\sum_{k\in\hat{K}_{rs}}\sum_{\substack{j\in\hat{K}_{rs}\\ j\neq k}} \beta_{kj}^{rs} f_{k(kj)}^{rs}\ln\frac{f_{k(kj)}^{rs}}{\beta_{kj}^{rs}} \quad \text{(Eq.3.1b)}$$

$$\tilde{z}_3 = \frac{1}{\theta}\sum_{r\in R}\sum_{s\in S}\sum_{k=1}^{|\hat{K}_{rs}|-1}\sum_{j=k+1}^{|\hat{K}_{rs}|} (1-\beta_{kj}^{rs})(f_{k(kj)}^{rs}+f_{j(kj)}^{rs})\ln\frac{f_{k(kj)}^{rs}+f_{j(kj)}^{rs}}{\beta_{kj}^{rs}} \quad \text{(Eq.3.1c)}$$

Subject to

$$\sum_{s\in S}\sum_{k\in\hat{K}_{rs}}\sum_{\substack{j\in\hat{K}_{rs}\\ j\neq k}} f_{k(kj)}^{rs} = O_r \quad \forall r\in R \quad \text{(Eq.3.2)}$$

$$\sum_{r\in R}\sum_{k\in\hat{K}_{rs}}\sum_{\substack{j\in\hat{K}_{rs}\\ j\neq k}} f_{k(kj)}^{rs} = D_s \quad \forall s\in S \quad \text{(Eq.3.3)}$$

$$f_{k(kj)}^{rs} \geq 0 \ \forall k\in\hat{K}_{rs}, kj\in\hat{L}_{rs}, r\in R, s\in S \quad \text{(Eq.3.4)}$$

It is noted that in CDA-PCL-SUE-R-PL only $f_{k(kj)}^{rs}$ are decision variables, since q_{rs} are substituted by $q_{rs} = \sum_{k\in\hat{K}_{rs}}\sum_{\substack{j\in\hat{K}_{rs}\\ j\neq k}} f_{k(kj)}^{rs}$. I next consider the following equivalent formulation to CDA-PCL-SUE-R-PL, which is the projection of CDA-PCL-SUE-R-PL onto the demand space, in order to solve the problem CDA-PCL-SUE-R-PL.

Equivalent Formulation to CDA-PCL-SUE-R-PL

$$\min U(q) \qquad \text{(Eq.4.1)}$$

Subject to

$$\sum_{s\in S} q_{rs} = O_r \quad \forall r \in R \qquad \text{(Eq.4.2)}$$

$$\sum_{r\in R} q_{rs} = D_s \quad \forall s \in S \qquad \text{(Eq.4.3)}$$

$$q_{rs} \geq 0 \ \forall r \in R, s \in S \qquad \text{(Eq.4.4)}$$

Where $U(q) = \tilde{z}_1 + \tilde{z}_2 + \tilde{z}_3$ (Eq.4.5)

Subject to

$$\sum_{k\in \hat{K}_{rs}} \sum_{\substack{j\in \hat{K}_{rs} \\ j\neq k}} f^{rs}_{k(kj)} = q_{rs} \ \forall r \in R, s \in S \qquad \text{(Eq.4.6)}$$

$$f^{rs}_{k(kj)} \geq 0 \ \forall k \in \hat{K}_{rs}, kj \in \hat{L}_{rs}, r \in R, s \in S \ \text{(Eq.4.7)}$$

This equivalent formulation utilizes the fact that the solution to Eq.4.5-4.7 (i.e. the restricted PCL-SUE) is easily obtained by the use of the PCL formula: $f^{rs}_{k(kj)} = P(kj)\cdot P(k\,|\,kj)\cdot q_{rs}$. By performing the substitution of the PCL formula in Eq.4.5 (i.e. $f^{rs}_{k(kj)} = P(kj)_n \cdot P(k\,|\,kj)_n \cdot$ q_{rs} and $f^{rs}_{j(kj)} = P(kj)_n \cdot P(j\,|\,kj)_n \cdot q_{rs}$ are substituted in $U(q) = \tilde{z}_1 + \tilde{z}_2 + \tilde{z}_3$), it can be proved that the implicit function $U(q)$ actually has the explicit form of the entropy maximization problem (problem Eq.5.1-5.8 in Phase I.1). Hence, it is clear that CDA-PCL-SUE-R-PL is solved through the solution of the entropy maximization problem followed by the application of the PCL formula. Specifically, an optimal solution to the equivalent formulation of CDA-PCL-SUE-R-PL is obtained by Phases I.1-I.3.

Phase I.1 Entropy Maximization Problem (the solution is denoted by $\underline{q}^n_{rs} \ \forall r \in R, s \in S$)

$$\min \sum_{r\in R}\sum_{s\in S} \gamma^n_{rs}\cdot \underline{q_{rs}} + \nu^n_{rs}\cdot \underline{q_{rs}}\cdot \ln \underline{q_{rs}} \qquad \text{(Eq.5.1)}$$

Subject to

$$\sum_{s\in S} \underline{q_{rs}} = O_r \quad \forall r \in R \quad :\alpha_r \qquad \text{(Eq.5.2)}$$

$$\sum_{r\in R} \underline{q_{rs}} = D_s \quad \forall s \in S \quad :\lambda_s \qquad \text{(Eq.5.3)}$$

$$\underline{q_{rs}} \geq 0 \ \forall r \in R, s \in S \qquad \text{(Eq.5.4)}$$

Where

$$\gamma^n_{rs} = \left\{ \begin{array}{l} \displaystyle\sum_{k\in \hat{K}_{rs}} \sum_{\substack{j\in \hat{K}_{rs} \\ j\neq k}} \left[\begin{array}{l} P(kj)_n \cdot P(k\,|\,kj)_n \cdot \\ \left(\begin{array}{l} c^{rs^n}_k + \dfrac{1}{\theta}\beta^{rs}_{kj}\cdot \\ \ln\left(\dfrac{P(kj)_n \cdot P(k\,|\,kj)_n}{\beta^{rs}_{kj}} \right) \end{array} \right) \end{array} \right] \\ + \displaystyle\sum_{k=1}^{|\hat{K}_{rs}|-1} \sum_{j=k+1}^{|\hat{K}_{rs}|} \frac{1}{\theta}(1-\beta^{rs}_{kj})\cdot P(kj)_n \cdot \ln \frac{P(kj)_n}{\beta^{rs}_{kj}} \end{array} \right\} \qquad \text{(Eq.5.5)}$$

$$\nu^n_{rs} = \left\{ \begin{array}{l} \displaystyle\sum_{k\in \hat{K}_{rs}} \sum_{\substack{j\in \hat{K}_{rs} \\ j\neq k}} \frac{1}{\theta}\beta^{rs}_{kj}\cdot P(kj)_n \cdot P(k\,|\,kj)_n \\ + \displaystyle\sum_{k=1}^{|\hat{K}_{rs}|-1} \sum_{j=k+1}^{|\hat{K}_{rs}|} \frac{1}{\theta}(1-\beta^{rs}_{kj})\cdot P(kj)_n \end{array} \right\} \qquad \text{(Eq.5.6)}$$

$$P(kj)_n = \frac{\beta^{rs}_{kj}\left(\exp(-\theta c^{rs^n}_k/\beta^{rs}_{kj}) + \exp(-\theta c^{rs^n}_j/\beta^{rs}_{kj})\right)^{\beta^{rs}_{kj}}}{\displaystyle\sum_{m=1}^{|\hat{K}_{rs}|-1}\sum_{l=m+1}^{|\hat{K}_{rs}|} \beta^{rs}_{ml}\left(\exp(-\theta c^{rs^n}_m/\beta^{rs}_{ml}) + \exp(-\theta c^{rs^n}_l/\beta^{rs}_{ml})\right)^{\beta^{rs}_{ml}}} \qquad \text{(Eq.5.7)}$$

$$P(k\,|\,kj)_n = \frac{\exp(-\theta c^{rs^n}_k/\beta^{rs}_{kj})}{\exp(-\theta c^{rs^n}_k/\beta^{rs}_{kj}) + \exp(-\theta c^{rs^n}_j/\beta^{rs}_{kj})} \qquad \text{(Eq.5.8)}$$

The entropy maximization problem Eq.5.1-5.8 can be solved by Bregman's balancing method (Lamond and Stewart, 1981; Bregman, 1967), and the result is an auxiliary demand $\underline{q}^n = \{\underline{q}_{rs}^n\}$. The balancing method is briefly described below.

Bregman's Balancing Method
Initialization of Balancing Method:

$\underline{q_{rs}^0} = \exp(-1 - \gamma_{rs}^n / \nu_{rs}^n) \quad \forall r \in R, s \in S$

(see *(21)* for the derivation of initial auxiliary O-D flows)

$t \leftarrow 0$ (t is iteration counter for the balancing method)

$i \leftarrow 1$ (i is the constraint counter of the entropy maximization problem)

General Step of Balancing Method (Balancing Constraint i):

Find the unique solution $\underline{q_{rs}^{t+1}}$ and ξ of:

$$\nu_{rs}^n \ln \underline{q_{rs}^{t+1}} - \nu_{rs}^n \ln \underline{q_{rs}^t} - \xi a_{i,rs} = 0 \;\; \forall r \in R, s \in S \qquad \text{(Eq.6)}$$

and $$\sum_{rs \in RS} a_{i,rs} \underline{q_{rs}^{t+1}} = b_i \,. \qquad \text{(Eq.7)}$$

The derivation of Eq.6 and Eq.7 is referred to Bregman (1967). Then, Eq.6 can be written as Eq.8.

$$\underline{q_{rs}^{t+1}} = \underline{q_{rs}^t} \exp\left(\frac{\xi a_{i,rs}}{\nu_{rs}^n}\right) \quad \forall r \in R, s \in S \qquad \text{(Eq.8)}$$

which is then substitute into Eq.(7), yielding:

$$\sum_{rs} a_{i,rs} \cdot \underline{q_{rs}^t} \exp\left(\frac{\xi a_{i,rs}}{\nu_{rs}^n}\right) = b_i \qquad \text{(Eq.9)}$$

where if $b_i = O_i$ and $\xi = \alpha_i$ if $1 \le i \le |R|$

$b_i = D_i$ and $\xi = \lambda_i$ if $|R|+1 \le i \le |R|+|S|$

ξ is determined by Newton's method, since it cannot be solved analytically.

Then, determine $\underline{q_{rs}^{t+1}}$ from Eq.8:

$i \leftarrow (i \text{ modulo} (|R|+|S|)) + 1$

$t \leftarrow t + 1$

For each pass of the algorithm (when all origins and destinations are balanced once), if $\underline{q_{rs}^t}$ is converged, terminate the algorithm.

<u>Phase I.2</u> The solution ($\underline{f}_{k(kj)}^{rs^n}$) of CDA-PCL-SUE-R-PL is obtained by applying the PCL formula:

$$\underline{f}_k^{rs^n} = P(k)_n \cdot \underline{q}_{rs}^n \;\; \forall k \in K_{rs}, r \in R, s \in S$$

Where

$$P(k)_n = \sum_{\substack{j \in \hat{K}_{rs} \\ j \ne k}} P(kj)_n \cdot P(k \mid kj)_n$$

<u>Phase I.3 (Line Search)</u>

An approximate line search is then made with respect to $z = z_1 + z_2 + z_3$ (the objective function of CDA-PCL-SUE-R) in the (feasible) direction of $\underline{f}^n - f^n$ and $\underline{q}^n - q^n$, resulting in the new solution f^{n+1} and q^{n+1}. Note that $\underline{f}^n$ and $\underline{q}^n$ are the auxiliary solutions to the auxiliary (partially linearized) problem CDA-PCL-SUE-R-PL; whereas f^n and q^n are the current solution to CDA-PCL-SUE-R at iteration *n*. The process is repeated with *n*=*n*+1 until a convergence criterion terminates the solution of CDA-PCL-SUE-R.

3.2. Phase II: Column Generation Problem

The partial linearization algorithm in Phase I solves the restricted master problem, given the subsets of routes between O-D pairs $\hat{K}_{rs} \subseteq K_{rs} \; \forall r \in R, s \in S$. The quality of travel pattern solution obtained from Phase I depends on the quality of $\hat{K}_{rs}$ in approximating K_{rs}. Damberg et al. (1996) suggested and evaluated two route generation strategies based on the calculation of shortest paths given the solution of the restricted master problem. I adopt Damberg et al.'s first route generation strategy for Phase II. Routes are generated from the solution of shortest path problems based on the deterministic travel times; i.e. random components of travel times are temporarily ignored. At the solution to this restricted master problem, the link travel times are updated accordingly, and the subsets $\hat{K}_{rs} \subseteq K_{rs}\ \forall r \in R, s \in S$ are augmented by the generation of new routes using the shortest path algorithm.

It is worth noting that the algorithm is not guaranteed to converge to the unique optimal solution of CDA-PCL-SUE. However, it is guaranteed to solve the restriction of CDA-PCL-SUE to any set of routes generated. In the proposed algorithm, it terminates when the root mean square error of link flows and O-D flows from two successive iterations are within a user-specified tolerance.

4. ILLUSTRATIVE EXAMPLES

The test network is a simple network with five nodes, eight links and four O-D pairs as shown in Figure 1. The Bureau Public Road link cost function is employed:

$$t_a(x_a) = t_a^0\left[1 + \alpha_a\left(\frac{x_a}{s_a}\right)^{\beta_a}\right]$$

The parameters t_a^0, s_a, α_a and β_a are also given in Table 1, and the length of link a is set to t_a^0. Two congestion levels are considered as follows. For higher-congestion level (lower-congestion level), origin demands of origin nodes 1 and 2 are 45 and 50 trips (22 and 25 trips), respectively; destination demands of destination nodes 4 and 5 are 35 and 60 trips (17 and 30 trips), respectively. The employed tolerances are $\varepsilon_{Simplicial} = \varepsilon_{Bregman} = \varepsilon_{Newton} = \varepsilon_{LineSearch} = 0.001$.

The CDA-MNL-SUE results are obtained from the algorithm in Lundgren and Patriksson (1998). The algorithms for both CDA-PCL-SUE and CDA-MNL-SUE are implemented in C. These run on a computer with 1.73 GHz Intel Core i7 processor and 4 GB of RAM, running under Windows 7. The CPU times of all runs on the test network are within 1 minute. I compare the results from CDA-PCL-SUE and CDA-PCL-MNL to examine the effects of congestion, travelers' stochastic perception error and path similarity to simultaneously solve doubly-constrained trip distribution problem and stochastic user equilibrium problem.

The dispersion parameters are set at various values for two congestion levels. The differences in O-D flows and link flows from the two combined distribution and assignment solutions is measured by the root mean square errors:

$$RMSE_L = \sqrt{\frac{1}{|A|}\sum_{a \in A}(x_{a,PCL}^* - x_{a,MNL}^*)^2}$$

Where $x_{a,PCL}^*$ and $x_{a,MNL}^*$ are the converged link flows in CDA-PCL-SUE and CDA-MNL-SUE, respectively.

$$RMSE_{OD} = \sqrt{\frac{1}{|RS|}\sum_{rs \in RS}(q_{rs,PCL}^* - q_{rs,MNL}^*)^2}$$

Where $q_{rs,PCL}^*$ and $q_{rs,MNL}^*$ are the converged O-D flows in CDA-PCL-SUE and CDA-MNL-SUE, respectively. Figure 2 shows the values of $RMSE_L$ and $RMSE_{OD}$ with various dispersion factors at two congestion levels. $RMSE_{OD}$ appears fluctuated at the higher-congestion level, whereas at the lower-congestion level $RMSE_{OD}$ appears smooth over the dispersion factors. At both congestion levels, $RMSE_L$ decreases with the increase of the dispersion factor. The decrease rate of $RMSE_L$ is greater when the dispersion factor is close to 0, and the decrease rate at the higher-congestion level is greater than that of the lower-congestion level on both networks. Based on the empirical results, the link flow patterns from CDA-PCL-SUE and CDA-MNL-SUE are closer as the dispersion factor increases on both congestion levels. The O-D flow patterns from both models differ in different degree over various dispersion factors.

Since the proposed algorithm employs the column generation phase to generate paths, it is possible that the generated paths from CDA-PCL-SUE are not the same as those from CDA-MNL-SUE. Then, it may not be comparable in terms of route flows. However, I found that the dispersion factor of 0.125 yields the same path set in both models. Thus, this is employed for path flow comparison. Table 2 shows the path flow results obtained from CDA-PCL-SUE and CDA-MNL-SUE at the higher-congestion level. As can be observed in Table 2, the path costs for each O-D pair in both CDA-PCL-SUE and CDA-MNL-SUE are not equal, and both models disperse travel demands to many paths for each O-D pair. These are the effects of travelers' stochastic perception error captured by both models. For each O-D pair, the similarity index is calculated for each route pair connecting this O-D pair. The similarity index of each route pair is completely independent of that of other route pairs. Prashker and Bekhor (2000) indicated that this property is highly desirable for route choice models. Table 2 shows the average similarity index for each route, which is the mean value of all similarity indices involving this route.

CDA-PCL-SUE generally considers a route with a high value of similarity as less attractive in route flow allocation. CDA-PCL-SUE accounts for the overlapping paths in route choice such that a path with a higher value of average similarity index and higher path cost will be assigned less flows. As can be seen in Table 2, in the CDA-MNL-SUE model, the cost of path 3 is 7.76% and 5.83% higher than paths 1 and 2, and assigns less flows to paths 3 (85.05% and 88.34% of flows assigned to paths 1 and 2, respectively). In contrast, CDA-PCL-SUE accounts for the path overlapping effect. The average similarity index of path 3 of O-D 1-4 is 101.09% higher than paths 1 and 2 connecting this O-D pair, and in the CDA-PCL-SUE model the cost of path 3 is 4.09% and 2.68% higher than paths 1 and 2. Then, CDA-PCL-SUE assigns much less flows to path 3 (51.82% and 53.24% of flows assigned to paths 1 and 2, respectively) than CDA-MNL-SUE does.

Table 3 shows the O-D flow results of the two models. Apparently, the O-D flows are distributed differently in the two models. As can be seen in Table 3, the total O-D flows out of each origin in both models are the same, and the total O-D flows into each destination in both models are equal. These are due to the doubly constrained trip distribution embedded in the two models. Table 3 also shows the average similarity index for each O-D pair, which is the mean value of the average similarity indices for all paths connecting this O-D pair. The weighted average path cost for each O-D pair is calculated by the summation of the products of path costs and route choice probabilities. I will explore the results to check whether I can relate the attractiveness of an O-D pair in doubly-constrained O-D trip distribution in CDA-PCL-SUE to the average similarity index for each O-D pair and the weighted average path cost of each O-D pair. I consider the O-D flow distribution for origin node 1. From Table 3, the weighted average path cost of O-D 1-4 in CDA-PCL-SUE is 19.78% higher than that of O-D 1-5, whereas in CDA-MNL-SUE it is 23.09% higher. The average similarity index of O-D 1-4 is 31.26% higher than O-D 1-5. The O-D flows allocated to O-D 1-5 is 14.00% higher than O-D 1-4 in CDA-PCL-SUE, whereas in CDA-MNL-SUE, it is 41.34% higher. It seems that CDA-PCL-SUE may assign more flows to O-D 1-4 with higher similarity index than CDA-MNL-SUE does. Next, I consider the O-D flow distribution for destination node 5. The weighted average path cost of O-D 1-5 in CDA-PCL-SUE is 45.99% higher than that of O-D 2-5, whereas in CDA-MNL-SUE it is 50.06% higher. The average similarity index of O-D 1-5 is 284.71% higher than O-D 2-5. The O-D flows allocated to O-D 2-5 is

50.29% higher than O-D 1-5 in CDA-PCL-SUE, whereas in CDA-MNL-SUE, it is 27.67% higher. In this case, CDA-PCL-SUE assigns less flow to O-D 1-5 with higher similarity index than CDA-MNL-SUE does. Apparently, it cannot be concluded how CDA-PCL-SUE distributes O-D flows among different O-D pairs, given weighted average path cost and average similarity index. This is because CDA-PCL-SUE also has the origin flow balance constraints and destination flow balance constraints that must be satisfied. In fact, the trip distribution in CDA-PCL-SUE can be determined by Eq.(2); i.e. it is based on the path costs, dispersion factor, dual variables of origin and destination flow balance constraints, and similarity indices. The average similarity indices and weighted average path costs are not directly employed in determining the trip distribution.

Table 4 shows the link flow results. The traffic flow patterns are different as the two models have different objective functions used in the trip distribution and route choice to capture the effects of congestion, stochastic perception error and path overlapping. Links with more paths passing through mostly have smaller flows assigned by CDA-PCL-SUE when compared with CDA-MNL-SUE such as links 1, 3, 4, 6 and 8. CDA-PCL-SUE assigns less number of flows to these links than CDA-MNL-SUE does.

5. SUMMARY AND CONCLUSIONS

The enhanced combined trip distribution and traffic assignment formulation is proposed. It combines the doubly-constrained gravity-model based trip distribution and the paired-combinatorial-logit stochastic user equilibrium assignment. The proposed solution method for CDA-PCL-SUE is a disaggregate simplicial decomposition algorithm. A test network with two congestion levels are employed. The results from CDA-PCL-SUE are compared to those from CDA-MNL-SUE in order to illustrate how CDA-PCL-SUE distributes O-D flows and route flows when accounting for similarity effects in addition to the congestion effect and stochastic-perception-error effect. I found that the relationship of O-D flow difference and dispersion factor is unclear, whereas link flow patterns from the two models are more identical at higher dispersion factors. CDA-PCL-SUE assigns less flow to a path with higher average similarity index and higher path cost than CDA-MNL-SUE. CDA-PCL-SUE generally assigns less flow to links with more paths passing through than CDA-MNL-SUE. The relationship between O-D flow allocation and the average similarity indices for O-D pairs is not obvious.

The future research is to include the singly-constrained gravity-based trip distribution version and to incorporate trip generation and modal split.

ACKNOWLEDGMENTS

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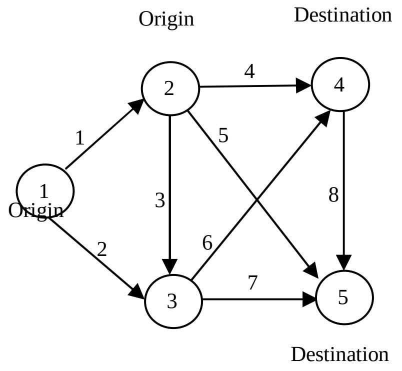

Figure 1 Test Network

Table 1 Parameters of Test Network

Link	s_a	t_a^0	α_a	β_a
(1,2)	25	4.0	0.15	4.0
(1,3)	25	5.2	0.15	4.0
(2,3)	30	1.0	0.15	4.0
(2,4)	15	5.0	0.15	4.0
(2,5)	15	5.0	0.15	4.0
(3,4)	15	4.0	0.15	4.0
(3,5)	15	4.0	0.15	4.0
(4,5)	30	1.0	0.15	4.0

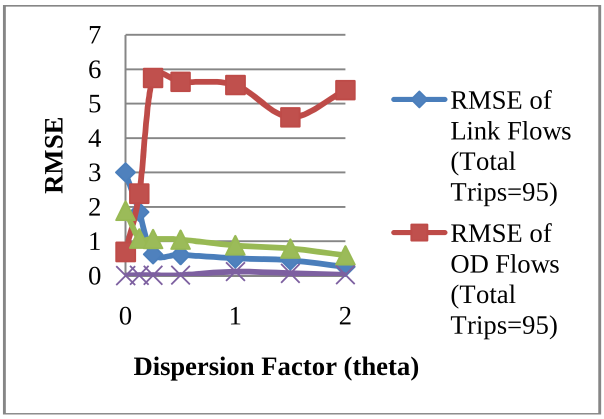

Figure 2 Root Mean Square Errors

Table 2 Path Flow Results of CDA-PCL-SUE and CDA-MNL-SUE (Total O-D Demand = 90 Trips, Dispersion Factor = 0.125)

O-D	Link Seq	Average Similarity Index	Route Choice Probability	
			PCL	MNL
1-4	1-4	0.2198	0.4014	0.3555
	2-6	0.2198	0.3907	0.3422
	1-3-6	0.4420	0.2079	0.3023
1-5	1-5	0.2811	0.2984	0.3250
	2-7	0.0000	0.4415	0.3632
	1-3-6-8	0.3072	0.1231	0.1433
	1-4-8	0.3072	0.1370	0.1685
2-4	4	0.0000	0.5204	0.5404
	3-6	0.0000	0.4796	0.4596
2-5	5	0.0000	0.3600	0.3393
	3-7	0.0609	0.3136	0.3350
	3-6-8	0.1164	0.1455	0.1497
	4-8	0.0556	0.1809	0.1760

O-D	Link Seq	Path Flow		Path Cost	
		PCL	MNL	PCL	MNL
1-4	1-4	8.43	6.62	15.95	16.68
	2-6	8.21	6.38	16.17	16.98
	1-3-6	4.37	5.63	16.61	17.97
1-5	1-5	7.17	8.56	12.17	12.45
	2-7	10.59	9.57	12.18	11.56
	1-3-6-8	2.93	3.77	17.63	19.00
	1-4-8	3.26	4.44	16.97	17.70
2-4	4	7.27	8.83	11.23	11.58
	3-6	6.70	7.51	11.88	12.88
2-5	5	12.99	11.41	7.45	7.36
	3-7	11.31	11.27	7.89	7.46
	3-6-8	5.22	5.03	12.90	13.90
	4-8	6.49	5.92	12.25	12.61

Table 3 O-D Flow Results of CDA-PCL-SUE and CDA-MNL-SUE (Total O-D Demand = 90 Trips, Dispersion Factor = 0.125)

O-D	Average Similarity Index	Weighted Average Path Cost		O-D Flow	
		PCL	MNL	PCL	MNL
1-4	0.2939	16.18	17.17	21.02	18.64
1-5	0.2239	13.50	13.95	23.97	26.35
2-4	0.0000	11.55	12.18	13.97	16.35
2-5	0.0582	9.25	9.30	36.02	33.64

Table 4 Link Flow Results of CDA-PCL-SUE and CDA-MNL-SUE
(Total O-D Demand = 90 Trips,
Dispersion Factor = 0.125)

Link	Number of Paths Passing Through*			
	O-D 1	O-D 2	O-D 3	O-D 4
1	2	3	0	0
2	1	1	0	0
3	1	1	1	2
4	1	1	1	1
5	0	1	0	1
6	2	1	1	1
7	0	1	0	1
8	0	2	0	2

Link	Link Flow		Link Cost	
	PCL	MNL	PCL	MNL
1	26.19	29.04	4.72	5.09
2	18.80	15.95	5.45	5.32
3	30.55	33.24	1.16	1.22
4	25.47	25.82	11.23	11.58
5	20.16	19.98	7.45	7.36
6	27.44	28.35	10.72	11.65
7	21.91	20.84	6.73	6.23
8	17.92	19.17	1.01	1.02

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THAI GUIDELINE FOR THE DESIGN OF SIGNAL CONTROL AT INTERSECTIONS – INTRODUCTION TO APPLIED PRELIMINARY CAPACITY ANALYSIS METHOD

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ABSTRACT:

The partner universities of the international scientific network «NICE on RoadS – EU-Asia Network in Competence Enhancement on Road Safety» developed a «Thai Guideline for the Design of Signal Control at Intersections». The guideline was prepared in the framework of the Thai-E.C. project «Improving Road Traffic Safety in Thailand – A Common Challenge for European and Thai Universities» based on international technology and knowledge transfer.

This paper gives an introduction to the Preliminary Capacity Analysis Method (PCA-Method), which is introduced in the developed guideline. The theoretical background of the PCA-Method and the related analysis steps will be introduced in this paper. Furthermore the application of the PCA-Method will be explained by a practical example.

KEY WORDS: signal control, road safety, preliminary capacity analysis

1. INTRODUCTION

The partner universities of the international scientific network «NICE on RoadS» developed a «Thai Guideline for the Design of Signal Control at Intersections» based on international technology and knowledge transfer (Vesper *et al.*, 2010b). The guideline was developed in the framework of the Thai-E.C. project «Improving Road Traffic Safety in Thailand – A Common Challenge for European and Thai Universities».

This paper gives an introduction to the Preliminary Capacity Analysis Method (PCA-Method), which is introduced in the developed guideline for preliminary capacity analysis of signal control at intersection.

The introduced PCA-Method is based on the German «AKF-Method» (Mueller, 1969). It is a simple and very effective method to estimate whether available capacity is sufficient to serve traffic volumes at signalised intersections or not.

2. DESIGN OF SIGNAL CONTROL

2.1 Control levels

The following two control levels can be distinguished in traffic control at signalised intersections:

- Area Control Level (macroscopic level),
- Local Control Level (microscopic level).

On the area control level, control strategies are defined which are effective for several signalised intersections located in an area, like e.g. in a road section, district or even in a whole city.

Whereas the local control level comprises signal control at a local single intersection. Here local control decisions are only effective for road users crossing the respective signalised intersection.

The prepared «Thai Guideline for the Design of Signal Control at Intersections» focuses mainly on the design of signal control on local control level. This means the design of signal control at local intersection.

2.2 Work-flow for the design of signal control

The design of signal control at local intersection can be subdivided in four design phases as introduced in the new «Thai Guideline for the Design of Signal Control at Intersections». Following this guideline the mentioned phases can be conducted successively in the design process of signal control as follows:

- Collection of Basic Design Data,
- Preliminary Capacity Analysis of Signal Control,
- Capacity Analysis of Signal Control,
- Final Check of Signal Control.

The work-flow for the design of signal control at local intersection (see Figure 1) gives an overview of the four design phases, the related sub-steps and the interrelation between them.

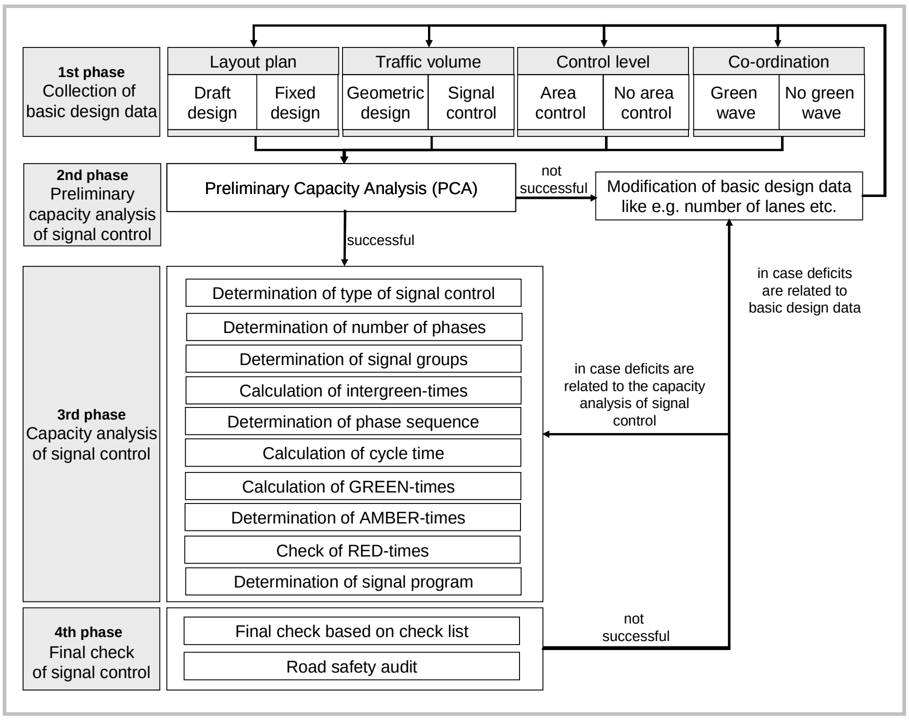


Figure 1 Work-flow for the Design of Signal Control at Local Intersection, (NICE on RoadS, 2011)

3. PRELIMINARY CAPACITY ANALYSIS

The Preliminary Capacity Analysis Method (PCA-Method) is applied in the 2nd phase of the work-flow for the design of signal control at local intersection as shown in Figure 1. Based on basic design data the PCA-Method can be applied to check whether the available capacity at signalised intersection is sufficient to serve design volumes or not. The PCA-Method, also known as AKF-Method (Mueller, 1969), can be subdivided in the following three calculation steps:

- Calculation of Available Capacity in Conflict Areas,
- Calculation of Required Capacity in Conflict Areas,
- Check of Sufficient Capacity.

In case available capacity is not sufficient, the basic design data needs to be changed (e.g. re-design of geometric design of intersection, such as: adding a lane, change the share of lanes, change the assignment of entry-lanes to signal groups, etc.). Also, other changes could be possible for example by forbidding selected traffic movements at a signalised intersection (e.g. forbidden right turning movements).

Results of preliminary capacity analysis can also show that existing road infrastructure is over-dimensioned. In this case, basic design data should also be changed but in the contrary direction (e.g. by reducing the number of entry-lanes or application of a more compact design of intersection).

3.1 Conflict areas

Traffic streams which cross a signalised intersection need to cross various conflict areas located in the inner area and in the exit-lanes of the intersection.

In Figure 2, the different conflict areas of a X-junction are marked and numbered as an example. The conflict areas 1 to 4 are located in the inner area of intersection, whereas the conflict areas 5 to 8 are located in the exit-lanes of the intersection.

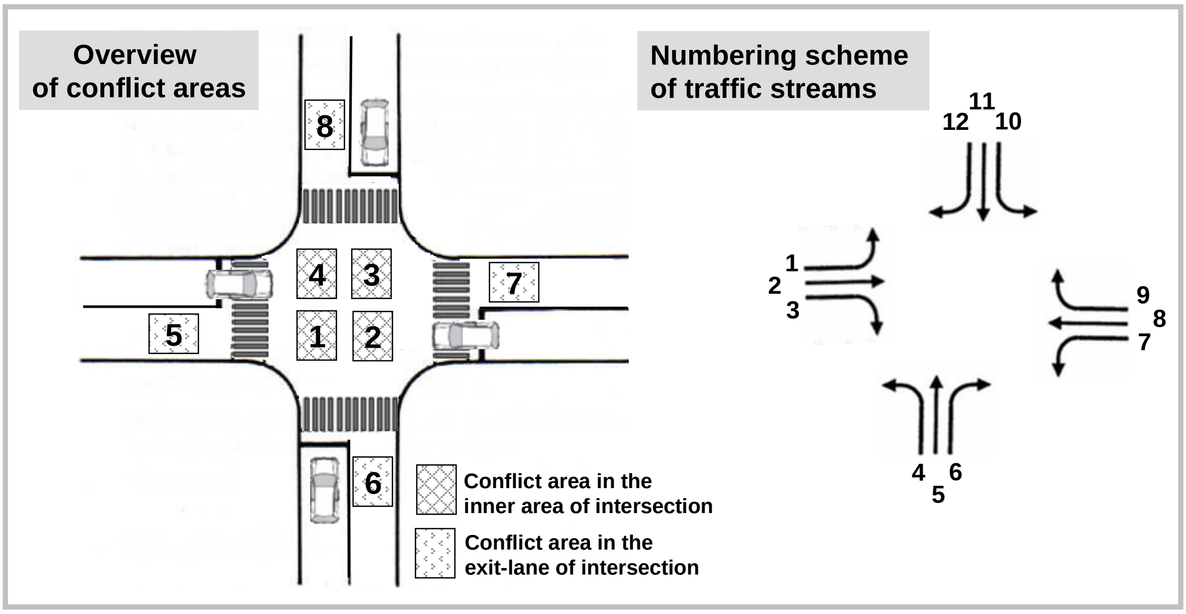


Figure 2 Conflict Areas and Numbering Scheme of Traffic Streams at a X-junction (NICE on RoadS, 2011)

3.2 Available capacity in conflict areas

The calculation of available capacity in conflict areas is the first calculation-step of PCA-Method.

The available capacity in conflict areas is limited and can be calculated as shown in Formula 1:

$$avail.\,C_{PCA} = \frac{3600 - \frac{3600}{t_C} * \sum t_{i,estimated}}{t_h} \qquad \text{(Eq.1)}$$

<u>legend:</u>

avail. C_{PCA}	: available capacity in conflict areas; [pcu/h]
t_C	: cycle time; [s/cycle]
$\Sigma t_{i,\ estimated}$	: total estimated intergreen-time per cycle; [s/cycle]
t_h	: mean time headway; [s/pcu]

For preliminary capacity analysis an estimated intergreen-time ($t_{i,\ estimated}$) of 6 to 7 s per phase-transition can be applied. Here one should consider that estimated intergreen-times are only applicable for preliminary capacity analysis and not for capacity analysis of signal control in the 3rd phase of the work-flow (see Figure 1).

For preliminary capacity analysis a mean time headway of 1,8 to 2,2 s per pcu can be applied. The mean time headway can differ significantly depending on the location and the geometric design of intersection. For this reason this parameter needs to be determined based on an onsite data inspection.

The choice of cycle time (t_C) has a strong influence on the available capacity in conflict areas (avail. C_{PCA}). The interrelation between the mentioned two parameters is shown in Figure 3 for different total intergreen-times per cycle. The diagram clarifies obviously that in case of low cycle times (up to ~70 seconds), the increase of cycle time has a great influence on available capacity in conflict areas at signalised intersections. In contrast, higher cycle times have an increasingly smaller influence on available capacity e.g. in a range over ~120 seconds.

For this reason it is recommended to apply cycle times in a range between 30 and 90 s as a regular solution and only in exceptional cases in a range between 90 up to 120 s. Cycle times higher than 120 s should be avoided.

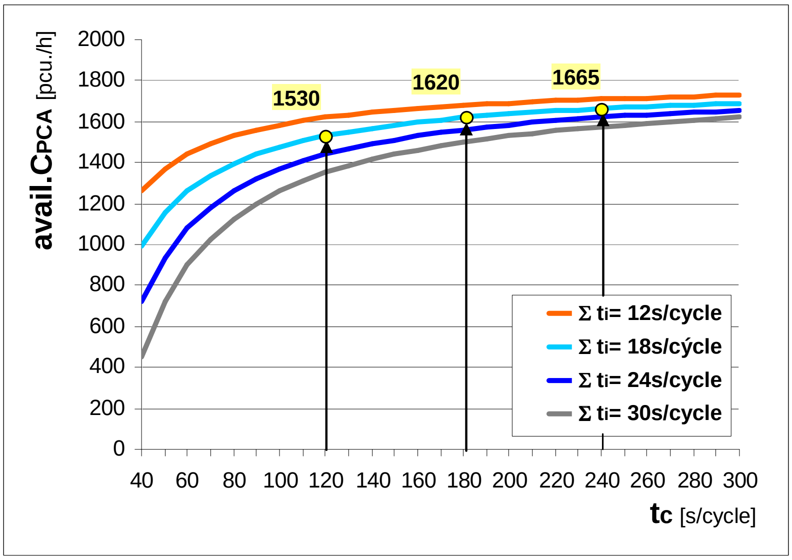


Figure 3 Interrelation between Cycle Time (t_c) and Available Capacity in Conflict Areas (avail. C_{PCA}) for a Mean Time Headway (t_h) of 2s/pcu; (Vesper *et al.*, 2008)

3.2.1 Practical example

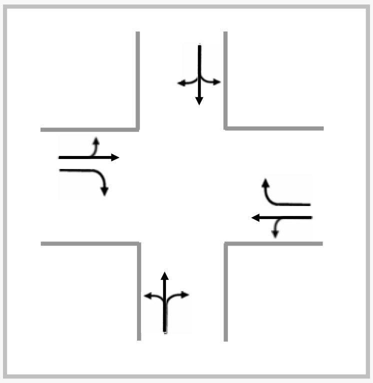
The application of the PCA-Method will be explained in a practical example (see Figure 4). The first calculation-step of PCA–Method is explained as an example in Figure 5.

The further calculation-steps of PCA-Method will be explained in the following subsections directly after the theoretical description of the next calculation steps.

Practical example: *Description*

Main design task: Check whether the available capacity of a X-junction is sufficient to cover the given design traffic volumes by application of the PCA-Method

Preconditions:
- X-junction belongs to an area control level with a cycle time of 90 s during morning peak-hour of a normal working-day,
- mean time headway of 2 s/pcu was calculated based on onsite data inspection,
- minimum GREEN-time of 10 s/cycle should be provided for all traffic streams,
- signal groups are not fixed,
- number and share of lanes is fixed:



- forcasted design traffic volumes:

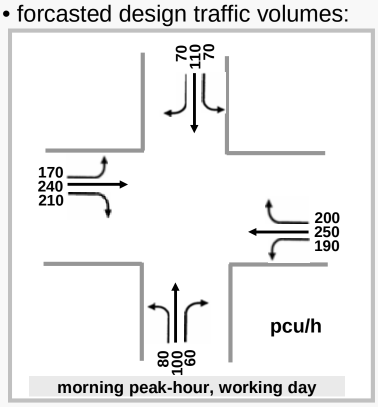


Figure 4 Application of PCA-Method, Description of Practical Example

***Practical example**: Calculation step 1*

Calculation of available capacity in conflict areas:

$$avail.C_{PCA} = \frac{3600 - \frac{3600}{t_C} * \sum t_{i,estimated}}{t_h}$$

t_c = 90 s/cycle (because intersection belongs to area control level, see *Figure 4*)

$\Sigma t_{i,\ estimated}$ = 4 phases/cycle * 6 s/phase-transition = 24 s/cycle

t_h = 2 s/pcu (according to *Figure 4*)

$$avail.C_{PCA} = \frac{3600 - \frac{3600}{90} * 24}{2} = 1320\ pcu/h$$

Figure 5 Application of PCA-Method, Calculation Step 1 of Practical Example

3.3 Required capacity in conflict areas

The calculation of required capacity in conflict areas is the second calculation-step of PCA-Method, which can be subdivided in the sub-steps “assignment of traffic streams” and “calculation of required capacity”.

3.3.1 Assignment of traffic streams

Traffic streams at signalised intersections need to cross conflict areas located in the inner area as well as in the exit-lanes of the intersection.

Each conflict area has to accommodate different traffic streams which want to cross the intersection as shown as an example in Figure 6.

The assignment of traffic streams to conflict areas at signalised intersection is based on the following two assignment-rules:

Rule 1: conflict areas in the inner area of an intersection: the two through traffic streams which cross each other in the conflict area and the respective opposing right turning traffic streams belong to the conflict area in the inner area of the intersection (e.g. in case of conflict area 3 in Figure 6: the two through traffic streams 2 and 11 and the respective opposing right turning traffic streams 6 and 9).

Rule 2: conflict areas in the exit-lanes of an intersection: the left turning, the through and the right turning traffic stream, all using the same exit-lane in order to leave the intersection belong to the respective conflict area in the exit-lane of the intersection (e.g. in case of conflict area 7 in Figure 6: the left turning traffic stream 10, the through traffic stream 2 and the right turning traffic stream 6).

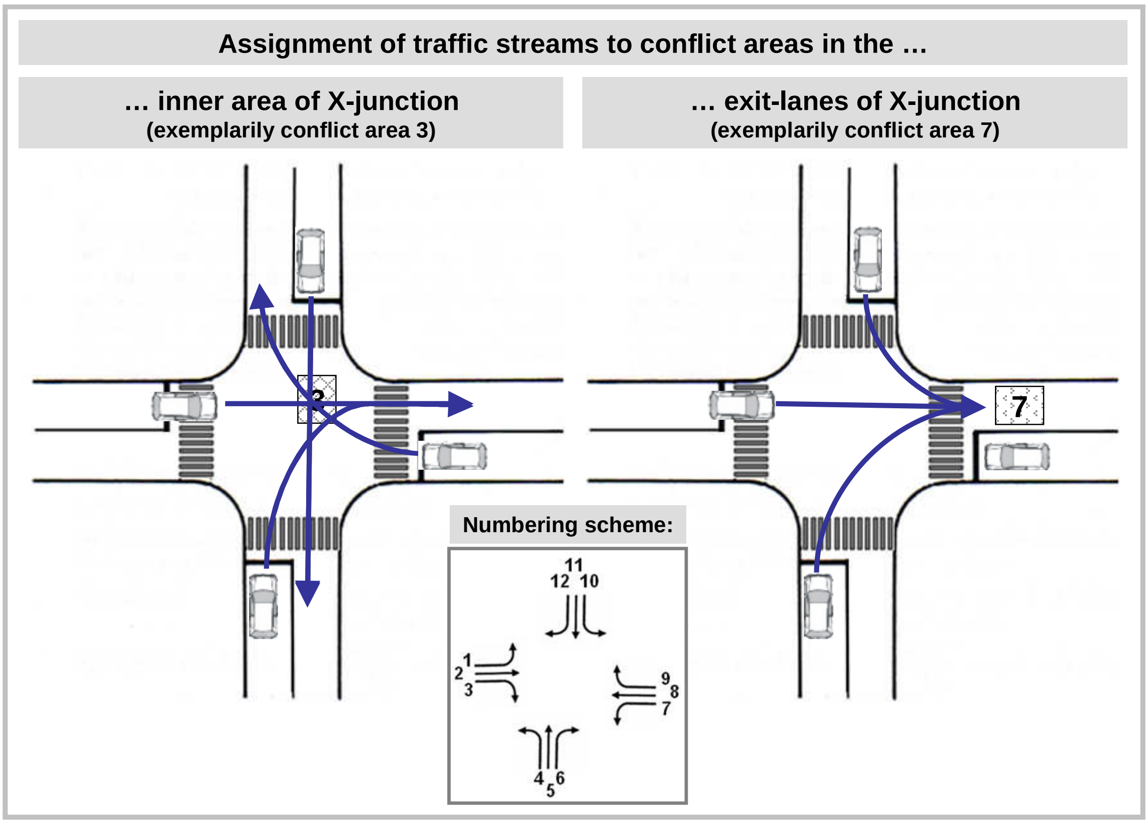
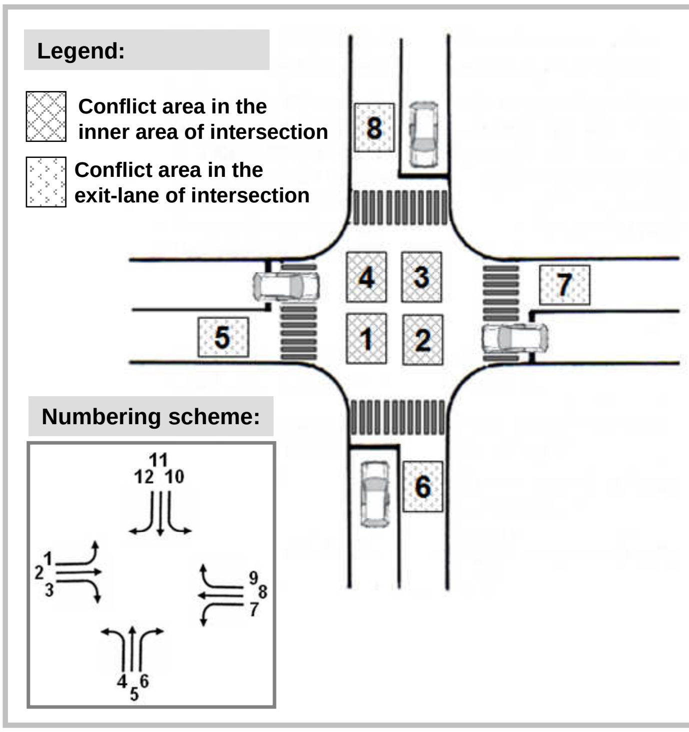


Figure 6 Assignment of Traffic Streams to Conflict Areas as an Example for Conflict Area 3 and 7; (NICE on RoadS, 2011)



Conflict area	Assigned traffic streams
1	5 + 12 + 8 + 3
2	8 + 3 + 11 + 6
3	11 + 6 + 2 + 9
4	2 + 9 + 5 + 12
5	4 + 8 + 12
6	7 + 11 + 3
7	10 + 2 + 6
8	1 + 5 + 9

Note: In case of a T-junction the missing conflict areas and traffic streams can be disregarded in the table above.

Figure 7 Assignment of Traffic Streams to Conflict Areas as an Example for a X-junction (NICE on RoadS, 2011)

In Figure 7 the different conflict areas and the assigned traffic streams are shown as an example for a X-junction. This table is also applicable for T-junctions. In case of a T-junction, the missing conflict areas and traffic streams need to be disregarded.

3.3.2 Calculation of required capacity

The required capacity in conflict areas depends:

- on the assigned traffic streams of conflict areas as shown in Figure 7
- and on the relevant design volume of traffic streams.

The relevant design volume of traffic streams (rel. V_i) can be determined based on basic design data and further requirements of signal control such as:

- design volume of traffic streams (V_i),
- share of lanes in the entries of intersection (exclusive or shared lanes; one or more lanes per direction)
- and the assignment of entry-lanes and related traffic streams to signal groups (here one should consider that one signal group can cover one or more entry-lanes, which can cover one or more traffic streams).
- The determination of relevant design volume of traffic streams is explained in Figure 8.

The required capacity in conflict areas can be calculated based on Formula 2 according to Figure 8.

$$req.C_{CAk} = \sum rel.V_{i,k} \qquad \text{(Eq.2)}$$

legend:

req. $C_{CA\,k}$: required capacity in conflict area k; [pcu/(h*conflict area)]

k : number of conflict area as shown in *Figure 7*; [-]

rel. V_i : relevant design volume of traffic stream i as introduced in *Figure 8*; [pcu/(h*entry-lane)]

Σrel. $V_{i,k}$: total amount of relevant design volumes of the traffic streams which are assigned to conflict area k as shown in *Figure 7*; [pcu/(h*conflict area)]

i : number of traffic stream according to numbering scheme as shown in *Figure 7*; [-]

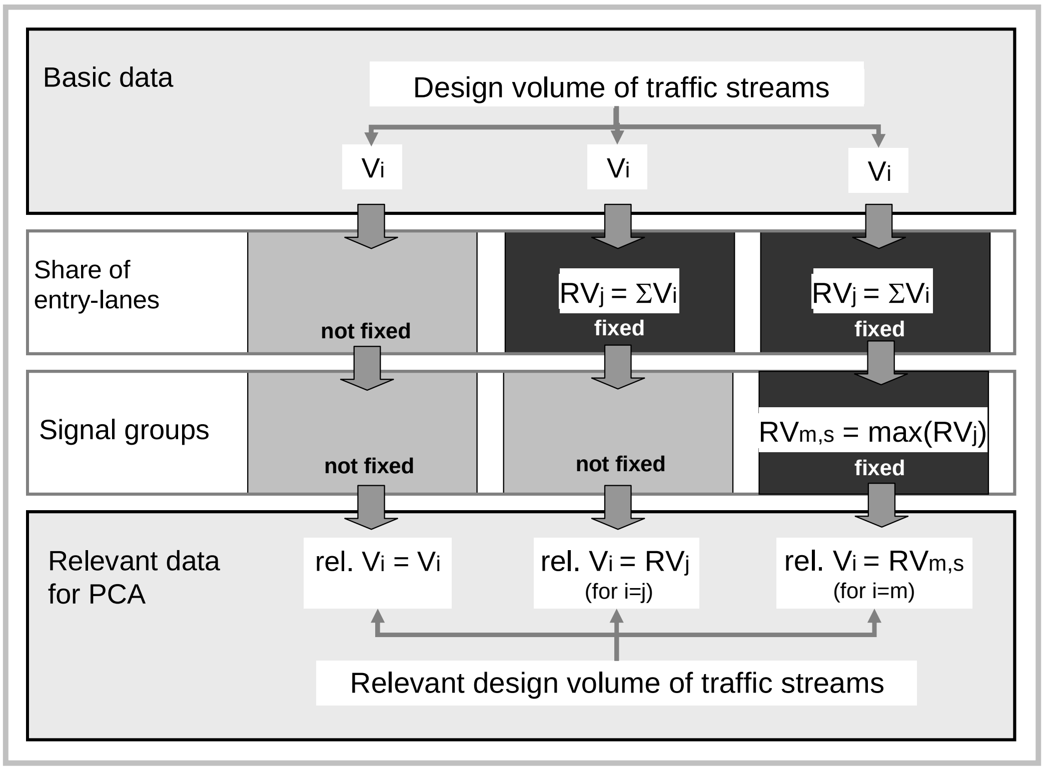


Figure 8 Determination of Relevant Design Volume of Traffic Streams (rel V_i) – overview (NICE on RoadS, 2011)

The main parameter in the calculation of the required capacity in conflict areas (req. $C_{CA\ k}$) is the relevant design volume of traffic streams (rel. V_i) at the considered intersection. The share of lanes and the assignment of traffic streams to signal groups need to be considered in the calculation of relevant design volume of traffic streams (rel. V_i) as introduced in Figure 8 and Formula 3.

$$rel.V_i = \begin{cases} V_i & \text{standard} \\ RV_j & \text{for i=j in case the share of entry-lanes is fixed} \\ RV_{m,s} & \text{for i=m in case the share of entry-lanes and the signal groups are fixed} \end{cases} \quad \text{(Eq.3)}$$

<u>legend:</u>

rel. V_i : relevant design volume of traffic stream i as introduced in *Figure 8*; [pcu/(h*entry-lane)]

V_i : design volume of traffic stream i; [pcu/(h*traffic stream)]

i : number of traffic stream according to numbering scheme as shown in Figure 7*;* [-]

RV_j : relevant design volume of entry-lane which covers traffic stream j; [pcu/(h*entry-lane)]

j : number of traffic stream according to numbering scheme as shown in Figure 7; [-]

$RV_{m,\,s}$: relevant design volume of signal groups which covers traffic stream m; [pcu/(h*entry-lane)]

s : signal group which covers traffic stream m; [-]

m : number of traffic stream according to numbering scheme as shown in Figure 7; [-]

Furthermore, it should be ensured that minimum GREEN-times (min t_{gr}) will be considered in the preliminary capacity analysis of signal control by the check that relevant design volume of traffic streams is higher than minimum relevant design volume of traffic streams (min rel. V_i) according to Formula 4:

$$rel.V_i \geq \min rel.V_i = \frac{3600}{t_c} * \frac{\min t_{gr}}{t_h} \quad \text{(Eq.4)}$$

<u>legend:</u>

rel. V_i : relevant design volume of traffic stream i as introduced in *Figure 8*; [pcu/(h*entry-lane)]

min rel. V_i : minimum relevant design volume of traffic stream i, considering minimum GREEN-time per cycle (min t_{gr});[pcu/(h*entry-lane)]

i : number of traffic stream according to numbering scheme as shown in Figure 7;[-]

t_c : cycle time; [s/cycle]

min t_{gr} : minimum GREEN-time which should be applied in signal program design; [s]

t_h : mean time headway; [s/pcu]

In case where the share of entry-lanes is pre-defined, and thereby fixed, the relevant design volume of entry-lane which covers the traffic streams (RV_j) needs to be considered in preliminary capacity analysis, as introduced in Figure 8 and Formula 3.

The relevant design volume of entry-lane which covers the traffic stream j (RV_j) can be calculated according to Formula 5.

$$RV_j = \sum V_i \quad \text{for traffic streams i which belong to the entry-lane which covers traffic stream j} \quad \text{(Eq.5)}$$

<u>legend:</u>

RV_j : relevant design volume of entry-lane which covers traffic stream j; [pcu/(h*entry-lane)]

j : number of traffic stream according to numbering scheme as shown in Figure 7; [-]

V_i : design volume of traffic stream i; [pcu/(h*traffic stream)]

i : number of traffic stream according to numbering scheme as shown in Figure 7; [-]

The calculation of the relevant design volume of entry-lanes (RV_j) is explained as an example in Figure 9.

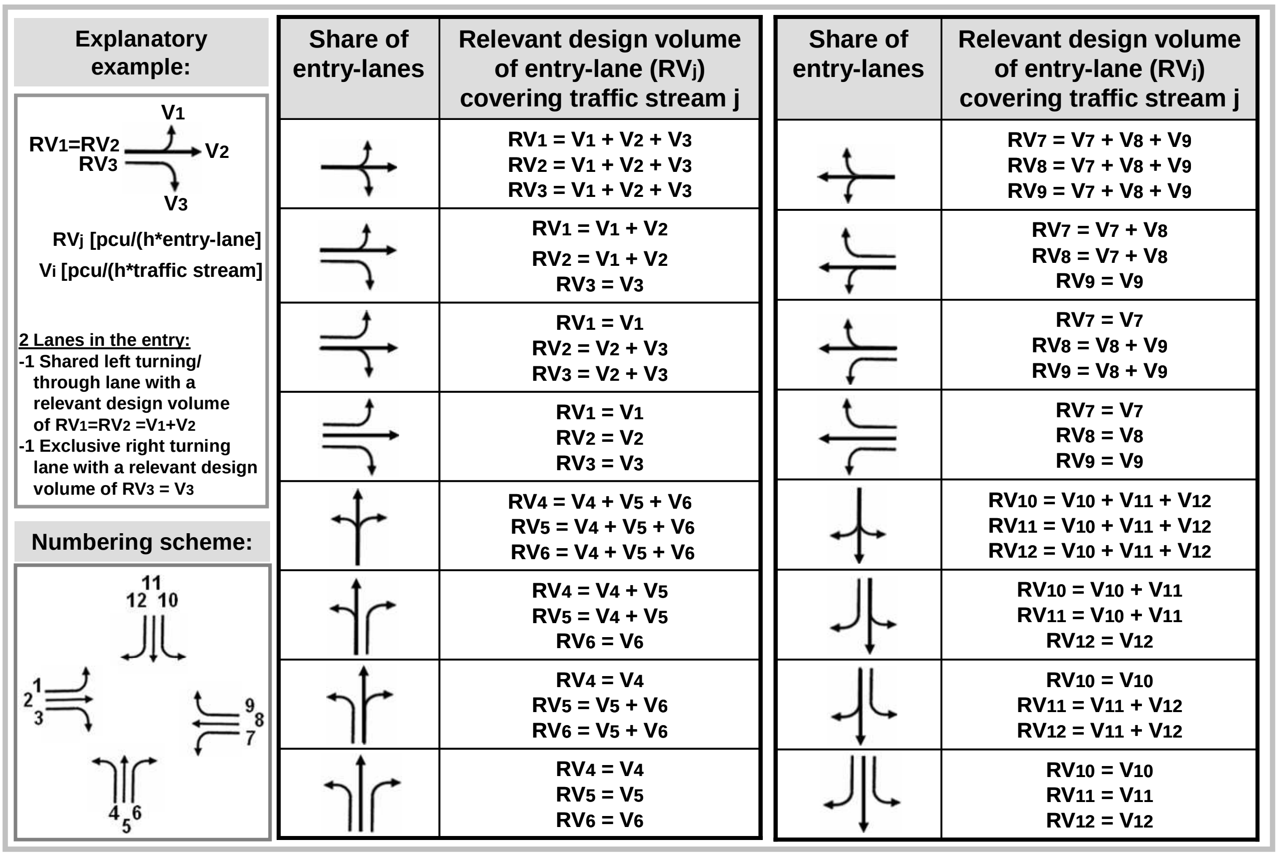


Figure 9 Relevant Design Volume of Entry-lanes (RV_j) and the Design Volume of Related Traffic Streams (V_i); (NICE on RoadS, 2011)

In case of high pedestrian traffic volumes, it should be proved whether design volumes V_i of turning traffic streams need to be increased because turning vehicles should give the priority to crossing pedestrians in the exit-lanes of intersection.

In case of two or more entry-lanes for one traffic stream, the split of traffic volumes between the two or more parallel lanes is relevant for calculation of the relevant design volume (RV_j) of each lane. For a first preliminary calculation, a proportional split is applicable in most cases.

In case where signal groups at signalised intersections are already fixed and cannot be changed, the relevant design volume of signal group s which covers traffic stream m ($RV_{m,s}$) needs to be considered in preliminary capacity analysis, as introduced in *Figure 8* and Formula 3.

Here, the relevant design volume of signal group s which can cover several traffic streams is equal to the maximum value of relevant design volumes of the covered entry-lanes (shared or exclusive lanes), as defined in *Formula 6.*

$$RV_{m,s} = \max\left(RV_j \right) \text{ for traffic streams j which belong to signal group s} \qquad \text{(Eq.6)}$$

legend:

$RV_{m,s}$: relevant design volume of signal group s which covers traffic stream m; [pcu/(h*entry-lane)];

s : number of signal group (e.g. V1, V2, …); [-]

m : number of traffic stream according to numbering scheme as shown in Figure 7; [-]

RV_j : relevant design traffic volume of entry-lane which covers traffic stream j; [pcu/(h*entry-lane)]

j : number of traffic stream; only traffic streams which are assigned to signal group s; [-]

3.3.3 Practical example

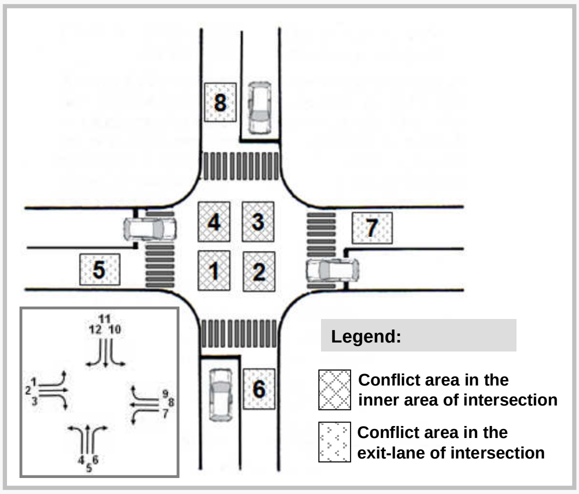
The application of second calculation-step of PCA-Method –the calculation of required capacity in conflict areas– is shown as an example in Figure 10 to 12.

***Practical example**: Calculation step 2 – part A*

Calculation of required capacity in conflict areas:

• *1st sub-step:* Assignment of traffic streams to conflict areas

Following the two assignment rules and the instructions in *Figure 7* the traffic streams at considered intersection can be assigned to the conflict areas 1 to 8 as listed in the table below.



Conflict area	Assigned traffic streams
1	5 + 12 + 8 + 3
2	8 + 3 + 11 + 6
3	11 + 6 + 2 + 9
4	2 + 9 + 5 + 12
5	4 + 8 + 12
6	7 + 11 + 3
7	10 + 2 + 6
8	1 + 5 + 9

Figure 10 Application of PCA-Method, Calculation Step 2 (part A) of Practical Example

<u>Practical example</u>: *Calculation step 2 – part B*

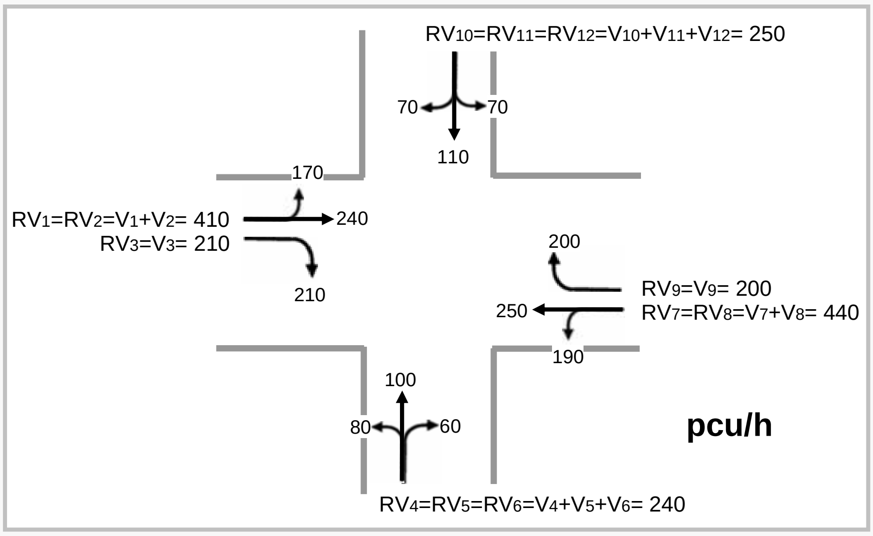
<u>Calculation of required capacity in conflict areas:</u>

• *2nd sub-step:* Calculation of relevant design volume of traffic streams (rel V_i):

• the number and share of lanes is fixed at considered intersection whereas the signal groups are not fixed as described in *Figure 4*; respectively the relevant design volume of traffic streams can be calculated according to *Formula 3* as follows*:*

$$rel.V_i = RV_j \quad \text{for i=j in case the share of entry-lanes is fixed}$$

• the relevant design volume of entry-lanes (RV_j) can be calculated according to *Figure 9* as shown below:



• a minimum GREEN-time of 10 s should be provided at the considered intersection. This requirement is fulfilled if the relevant design volume of traffic streams at the considered intersection is higher than a minimum value as introduced in *Formula 4*:

$$rel.V_i \geq \min rel.V_i = \frac{3600}{t_c} * \frac{\min t_{gr}}{t_h} = \frac{3600}{90} * \frac{10}{2} = 200 \ pcu/h$$

requirement ist fulfilled for all traffic streams !

• finally the relevant design volumes of traffic streams can be summarised as follows:

rel. V_1 = RV_1 = 410 pcu/h rel. V_2 = RV_2 = 410 pcu/h rel. V_3 = RV_3 = 210 pcu/h	rel. V_4 = RV_4 = 240 pcu/h rel. V_5 = RV_5 = 240 pcu/h rel. V_6 = RV_6 = 240 pcu/h
rel. V_7 = RV_7 = 440 pcu/h rel. V_8 = RV_8 = 440 pcu/h rel. V_9 = RV_9 = 200 pcu/h	rel. V_{10} = RV_{10} = 250 pcu/h rel. V_{11} = RV_{11} = 250 pcu/h rel. V_{12} = RV_{12} = 250 pcu/h

Figure 11 Application of PCA-Method, Calculation Step 2 (part B) of Practical Example

<u>Practical example</u>: *Calculation step 2 – part C*

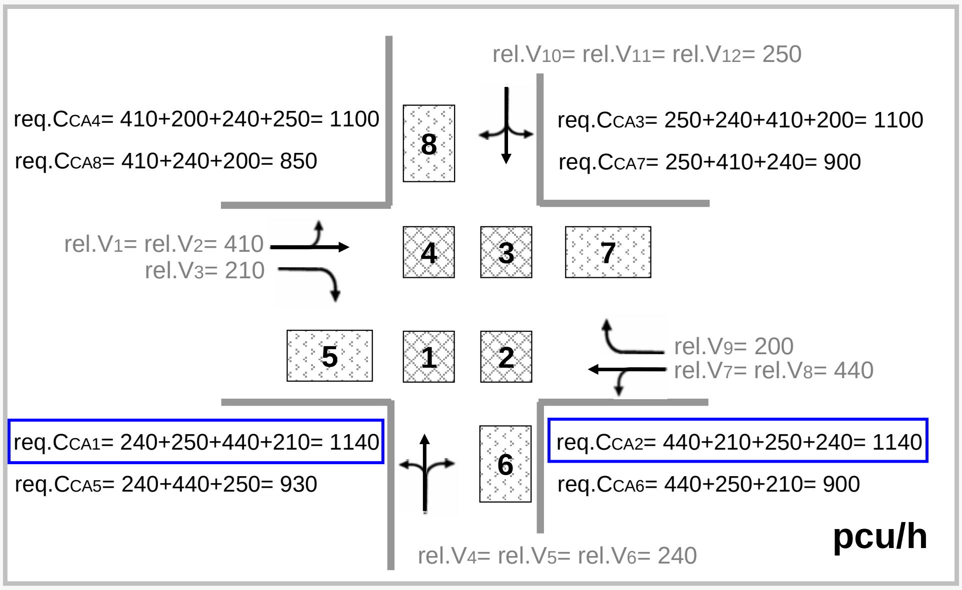
<u>Calculation of required capacity in conflict areas:</u>

- *3rd sub-step:* Calculation of required capacity in conflict areas (req. C_{CAk}):

  - the required capacity in conflict areas (req. C_{CAk}) can be calculated based on *Formula 2 as follows*:

$$req.C_{CAk} = \sum rel.V_{i,k}$$

  - the required capacity in conflict areas (req. C_{CAk}) can be calculated based on assigned traffic streams to conflict areas (see *Figure 10*) and related relevant design volume of traffic streams (rel Vi; see *Figure 11*) as shown below:



➔ the maximum required capacity in conflict areas is required in conflict areas 1 and 2 as marked in the figure above.

Figure 12 Application of PCA-Method, Calculation Step 2 (part C) of Practical Example

3.4 Check of sufficient capacity

The check of sufficient available capacity can be conducted based on the calculated available capacity in conflict areas and the required capacity in conflict areas.

The check of sufficient capacity can be subdivided into the following two work-steps:

- Calculation of Available Degree of Saturation,
- Capacity Check.

3.4.1 Available degree of saturation

The available degree of saturation (ADS) is a value which describes which part of available capacity will be used by road users.

The available degree of saturation (ADS) can be calculated by using Formula 7*:*

$$ADS_k = \frac{req.C_{CA\,k}}{avail.C_{PCA}} \qquad \text{(Eq.7)}$$

legend:

ADS_k : available degree of saturation in conflict area k; [-]
req. $C_{CA\,k}$: required capacity in conflict area k; [pcu/(h*conflict area)]
avail. C_{PCA} : available capacity in conflict areas; [pcu/(h*conflict area)]
k : number of conflict area as shown in Figure 7; [-]

3.4.2 Capacity check

Because of randomness of arriving vehicles in the entries of signalised intersections, the target degree of saturation (TDS) in the design of signal control should be at maximum:

$$TDS = \begin{cases} \leq 0.85 \text{ to } 0.90 & \text{in case of not co-ordinated intersections} \\ \leq 0.80 \text{ to } 0.85 & \text{in case of co-ordinated intersections} \end{cases} \quad \text{(Eq.8)}$$

legend:

TDS : target degree of saturation [-]

Following this requirement, a signalised intersection is said to provide sufficient capacity if the requirement in *Formula 9* is fulfilled:

$$ADS_k \leq TDS \quad \text{(Eq.9)}$$

legend:

ADS_k : available degree of saturation in conflict area k; [k]
k : number of conflict area as shown in Figure 7; [-]
TDS : target degree of saturation [-]

The limiting TDS (see Formula 8) ensures that approaching road users will be served at signalised intersection within cycle time. Furthermore the limiting TDS for co-ordinated intersections ensures that progression speed can be maintained for co-ordinated traffic streams.
In case where the main parameters of the preliminary capacity analysis will also be applied in the design of signal program, it can be expected that the detailed signal program calculation will be successful too.

The main parameters of the preliminary capacity analysis are amongst others:

- Design of Intersection (number and share of entry-lanes (shared/exclusive)),
- Assignment of Entry-lanes to Signal Groups,
- Cycle Time (t_c),
- Target Degree of Saturation (TDS).

If the available capacity is not sufficient, the basic design data needs to be modified or changed (see work-flow in Figure 1*)*. Based on changed basic design data, the preliminary capacity analysis can be conducted again in order to prove that the available capacity is sufficient under the changed condition.

3.4.3 Practical example

The application of third calculation step of PCA-Method –the check of sufficient capacity– is shown as an example in Figure 13.

***Practical example**:* *Calculation step 3*

Check of sufficient capacity:

- the maximum required capacity in conflict areas is required in conflict area 1 and 2 as shown in *Figure 12*. If the capacity check is successful for these two conflict areas, the check will be also successful for the other conflict areas.

- the available degree of saturation can be calculated for conflict area 1 and 2 based on *Formula 7*:

$$ADS_1 = \frac{req.\,C_{CA1}}{avail.\,C_{PCA}} = \frac{1140}{1320} = 0{,}86 \qquad ADS_2 = ADS_1 = 0{,}86$$

- the capacity check is successful in case the requirement of *Formula 9* is fulfilled:

$ADS_1 = ADS_2 \leq TDS_{not\ co-ordinated}$ ➔ $0{,}86 \leq 0{,}85 - 0{,}90$ not co-ordinated intersection

➔ the capacity check is successful for conflict area 1 and 2
➔ the capacity check is also successful for other conflict areas, because their ADS_k is lower than in the conflict areas 1 and 2.

Figure 13 Application of PCA-Method, Calculation Step 3 of Practical Example

4. CONCLUSION

The theoretical background of the Preliminary Capacity Analysis Method (PCA-Method) is introduced in this paper in detail. Furthermore the different calculation steps of the PCA-Method are explained for a signalised X-junction as a practical example.

Beside the introduced PCA-Method the new developed «Thai Guideline for the Design of Signal Control at Intersections» comprises further main parts, which should be applied in the design of signal control. These are the “Collection of Basic Design Data”, the “Capacity Analysis of Signal Control” and the “Final Check of Signal Control” as introduced in the work-flow in Figure 1.

Altogether the new developed «Thai Guideline for the Design of Signal Control at Intersections» provides a set of design instructions and recommendations which will help Thai engineers and practitioners to improve signal control at signalised intersections in the future.

It is expected that improved signal control will contribute to safer and more efficient use of signal control at intersections and thereby to contribute to improve road traffic safety as well as quality flow in the road network.

5. FORECAST

The partner universities of the international network «NICE on RoadS» prepared the «Thai Guideline for the Design of Signal Control at Intersections» in the framework of of the Thai-E.C. project «Improving Road Traffic Safety in Thailand – A Common Challenge for European and Thai Universities».

The guidelines will be published as an English version very soon and will be ready for application in practice afterwards.

Furthermore the Thai partner universities will develop a Thai version of the guideline by the end of 2011. In this way the guideline will also be available in Thai language.

In May 2011 the partner universities of «NICE on RoadS» established the "EU-Asia Road Safety Centre of Excellence – RoSCoE", which is located on the Hat Yai campus of the Prince of Songkla University in the south of Thailand.

Here 6 PhD scholars are conducting their PhD-study, dealing with several road safety related aspects of road design. It is planned that they summarise results of their studies in design guidelines. In this way further design guidelines will be available for Thai engineers and practitioners for application in the future.

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