Thermal Stress and Deformation of Steel Structures of High Rise Buildings in Fire

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ABSTRACT

In analyzing thermal stresses and deformation of a high rise steel structure, exposed to a compartment fire, it is convenient to divide the structure into (a) the local substructure, directly fire exposed, (b) the adjacent substructure, comprising the nearest floors and spans around the fire compartment, and (c) the remaining part of the structure.
For substructure (a), the deformation behavior is normally elasto-plastic, for substructure (b) purely elastic. For the remaining part (c), the fire induced stresses and deformations are generally negligible.
This paper presents a method for the calculation of the stress and deformation behaviour of a high rise steel structure, exposed to a compartment fire, based on a subdivision of the structure as above. In order to illustrate the structural fire behaviour, 48 buildings have been analysed according to the method presented.

Keywords thermal stress,thermal deformation,steel structure,plastic hinge,
          structural stability, yield temperature, structural divide

1. INTRODUCTION

Fire engineering design of building structures and structural members is generally, in Japan, carried out based on the results of standard fire resistance tests. The object of this design is to ensure the safety of structural members which are simply supported and bearing the maximum allowable service loads. Therefore, fire engineering design according to current laws and specifications gives a lower fire resistance than that of main structural members which are designed against seismic loads. Rational fire design of building structures can be made by analyzing the thermal stresses and deflections of building structures on the basis of temperature distribution of structural members and fire behavior in fire compartment.
Deflection behavior of a structural frame, exposed to fire, depends upon loading, reduction of load bearing capacity and stiffness and thermal stresses due to elevated temperature. Reductions of load bearing capacity have been thoroughly studied experimentally and experimental formulations have been suggested for elastic modulus, yield point and buckling stress of steel at elevated temperature/1-5/. The development of thermal stresses in steel structural members,generally, depends on end restraint conditions

against thermal expansion and mechanical characteristics of steel at elevated temperature.

Prediction methods of steel structure and structural members have been developed rather earlier in elastic range /7,10,15/ and later also in elasto-plastic range /6,8,9,11,17-23/. These methods,however,are primarily intended to be applied to multi-story frames because of limitation on computer capacity and are not well adapted to high rise building structures.

The distribution and development of thermal stresses in high rise steel structure, exposed to the compartment fire,is localized around this compartment. Therefore, in analyzing thermal stresses and deformation of a high rise steel structure,exposed to a compartment fire,it is convenient to divide the structure into (a) the local substructure,directly fire exposed, (b) the adjacent substructure, comprising the nearest floors and spans around the fire compartment, and (c) the remaining part of the structure. For substructure (a), the deformation behavior is normally elasto-plastic, for substructure (b) purely elastic. For the remaining part (c), the fire induced stresses and deformations are generally negligible /10,14/.

In this paper, we suggest a method of calculation of the restrained forces acting upon the local substructure and related deflections of the adjacent substructure. The determination of the restraint forces accurately takes into account the stiffness of and the forces within the total structure. The deflection of the adjacent substructure can rapidly be further transferred from one of the local substructure.

Using this calculation method, fire response is examined for 48 buildings, exposed to compartment fires.

2. CALCULATION METHOD

For the analysis of the state of thermal stresses and deflection at a compartment fire exposure, the steel structure is divided into the local substructure, the adjacent substructure and the surrounding substructure according to Fig.1. Thermal stresses and deflections are of importance for the local and adjacent substructures and practically negligible for the surrounding substructure. The analysis includes the influence of material and geometrical non-linearities. The local substructure is significantly affected by the fire exposure and must be analyzed in elasto-plastic range. The adjacent substructure is not affected directly by the fire exposure and behaves normally in elastic range. The equilibrium equation for the local substructure and the compatibility equation between integrated and incremental deformations are formulated as follows.

$$
\begin{vmatrix} R^i - {}_T P_R^{i,j} \\ P_L - {}_T P_L^{i,j} \end{vmatrix} = \begin{vmatrix} {}_T K_{RR}^{i,j} & {}_T K_{RL}^{i,j} \\ {}_T K_{LR}^{i,j} & {}_T K_{LL}^{i,j} \end{vmatrix} \cdot \begin{vmatrix} \Delta U_R^{i,j+1} \\ \Delta U_L^{i,j+1} \end{vmatrix} \tag{1}
$$

$$
U_R^{i,j+1} = U_R^{i,j} + \Delta U_R^{i,j+1} \tag{2}
$$

where capital letter **R, P, K, U** mean end restraint force, external joint force, stiffness of substructure, displacement of joint,respectively. The subscript letter R, L mean that the quantity is related to the joint acted upon by the restraint force and the joint defined by segmentation of member in local substructure, respectively, and the subscript letter T to the influence of temperature. The subscript i and j mean time step and iterative step for approach to nonlinear phenomenon using linear relationship, respectively. ${}_T P_R^{i,j}$, ${}_T P_L^{i,j}$ is joint force which is calculated by integration of interior stress. This stress is determined from stress-strain curve and history and instantaneous stress-related
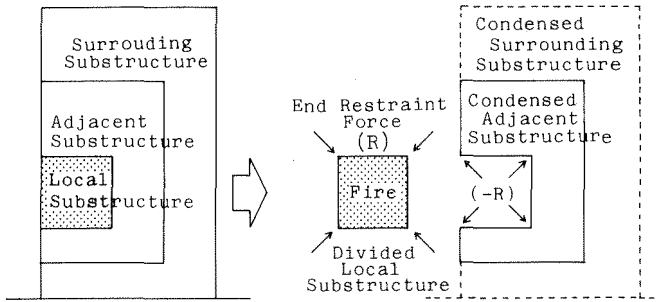
Fig.1 Division of Structure

strain is derived with subtracting thermal and creep strains from total strain. Therefore, the influence of instantaneous stress strain curve, thermal strain and creep strain dependent on time and temperature is fully included in the joint force $_T P_R^{i \cdot j}$, $_T P_L^{i \cdot j}$.

The force equilibrium equation at joints in the adjacent and surrounding substructure is assembled, considering the end restraint force **R** reversely, acting upon the boundary joints between the local and adjacent substructure, as follows.

$$
\begin{vmatrix} P_R - R^i \\ P_A \\ P_G \end{vmatrix} = \begin{vmatrix} K_{RR} & K_{RA} & O \\ K_{AR} & K_{AA} & K_{AG} \\ O & K_{GA} & K_{GG} \end{vmatrix} \cdot \begin{vmatrix} U_R^{i \cdot j+1} \\ U_A \\ U_G \end{vmatrix} + \begin{vmatrix} _0 P_R \\ _0 P_A \\ _0 P_G \end{vmatrix} \quad\text{——— (3)}
$$

where $_0 P$ is equivalent joint force due to intermediate loads acting on members in the adjacent and surrounding substructure. The subscript letter **A, G** mean quantities related to the adjacent and surrounding substructure, respectively. The stiffness matrix K_{GG}, K_{GA} and joint force vector $_0 P_G$, P_G of the surrounding substructure are condensed and related only to the boundary joints between the local and adjacent substructure. The practical procedure of condensation is, firstly, upper back step elimination of K_{GG} and secondary, unitization of K_{AA} by square elimination and then the force equilibrium equation (3) are condensed in connection with the boundary displacement vector $U_R^{i \cdot j+1}$,

$$
\begin{vmatrix} P_R - R^i \\ P_A' \\ P_G' \end{vmatrix} = \begin{vmatrix} K_{RR}' & O & O \\ K_{AR}' & I & O \\ O & K_{GA}' & K_{GG}' \end{vmatrix} \cdot \begin{vmatrix} U_R^{i \cdot j+1} \\ U_A \\ U_G \end{vmatrix} + \begin{vmatrix} _0 P_R' \\ _0 P_A' \\ _0 P_G' \end{vmatrix} \quad\text{——— (4)}
$$

The end restraint force vector R^i is derived from the first line of the condensed equation (4), as follows.

$$
R^i = -K_{RR}' \cdot U_R^{i \cdot j+1} - \{ _0 P_R' - P_R' \} \quad\text{————————— (5)}
$$

The end restraint force vector R^i is formulated in terms of the

condensed stiffness matrix K_{RR}' solely related to the boundary joint displacement vector $U_R^{i,j+1}$ and the transferred force vector $\{\,_0 P_R' - P_R'\,\}$. Substituting equation (5) in (1) and using the displacement compatibility equation (2), following equation is derived:

$$\left|\begin{array}{l} P_R' - \,_0 P_R' - K_{RR}' \cdot U_R^{i,j} - \,_T P_R^{i,j} \\[2mm] P_L - \,_T P_L^{i,j} \end{array}\right|$$

$$= \left|\begin{array}{cc} \,_T K_{RR}^{i,j} + K_{RR}' & \,_T K_{RL}^{i,j} \\[2mm] \,_T K_{LR}^{i,j} & \,_T K_{LL}^{i,j} \end{array}\right| \cdot \left|\begin{array}{c} \Delta U_R^{i,j+1} \\[2mm] \Delta U_L^{i,j+1} \end{array}\right| \quad\text{------------------} \quad (6)$$

Calculation of incremental displacements $\Delta U_R^{i,j+1}$, $\Delta U_L^{i,j+1}$ is carried out iteratively according to Newton method and these values are converged to zero in the local substructure at each time step.

The adjacent substructure joint displacement vector is derived from the second line of condensed equation (4) as follows.

$$U_A = -K_{AR}' \cdot U_R^{i,j+1} + \{\,P_A' - \,_0 P_A'\,\} \quad\text{--------------}\quad (7)$$

where the boundary displacement vector $U_R^{i,j+1}$ is obtained from the analysis of the local substructure, considering the compatibility equation (2).

3. NUMERICAL ANALYSIS

    The thermal stress analysis of high rise buildings is demonstrated in the following, assuming a fire in some compartment. The local substructure which must be analyzed in elasto-plastic range is shown in Fig.2 as "H" shaped frame which includes upper columns adjacent to fire exposed members. This substructure is analyzed, separated from the whole structure and acted on by restraint forces. Adjacent substructure occupies 6 stories, containing upper and lower 2 stories of the local substructure.
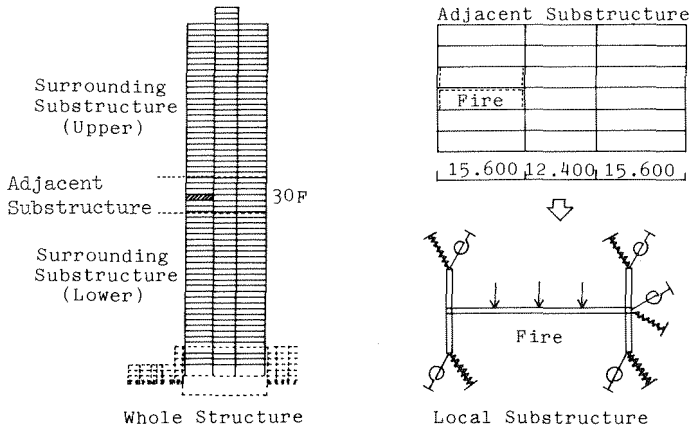

Fig.2 The Outline and Division of Real Structure (Example A)

Thermal stress analysis of the local substructure is carried out by using "a non linear direct stiffness formulation coupled with a time step integration" /13/. The assumptions of the analysis are as follows.

(1) Time dependent temperature variation is assumed step-wise and temperature distribution of the member remains constant within a given time step.

(2) Beams and columns which constitute the local substructure are divided longitudinally into segments. In the example referred, beams are divided into 12 segments and columns into 7 segments.

(3) Segments are further divided into subslices within the cross section; in the example, flange into 2 subslices and web into 6 subslices.

(4) Strain distribution remains plane according to Bernoulli's hypothesis.

(5) Total strain ε in segment consists of instantaneous stress-strain ε^S and thermal expansion strain ε^T. Creep strain ε^C is neglected for simplification.

$$\varepsilon = \varepsilon^S + \varepsilon^T$$

(6) Instantaneous stress-strain is defined as follows /16/.

$$\varepsilon^S = \varepsilon^e + \varepsilon^P = \sigma / {}_0E_T + |\sigma / \sigma_k|^k \, \text{sgn}(\sigma)$$

where ε^e = elastic strain, ε^P = plastic strain, σ = stress, ${}_0E_T$ = initial modulus of elasticity and σ_k, k are quantities which determine plastic strain from stress-strain curve.

(7) Thermal strain is defined as follows /12,13/.

$$\varepsilon = \alpha T = (10.8 + 0.00675T) \, T \times 10^{-6}$$

48 buildings were analyzed whose structural data were available and which had been permitted to be constructed by 1980. These are listed in Table 1. The buildings are grouped into 7 types according to location and size of fire compartment in a floor. The compartment fire is assumed to be Standard Fire.

The computational analysis was carried out for the building structure shown in Fig. 2 (type A, building No.43 in Table 1), supposing compartment fire on 30th floor. Results are given in Fig. 3 ∿ 6. As shown in Fig. 3, development of the bending moment is remarkable at the top and bottom of the columns due to the thermal elongation of the beam exposed to fire. Fig.4 shows that the bending moments increase to a maximum at about 400°C, and then decrease following to reduction of load bearing capacity. As shown in Fig.5, horizontal displacement at top of the outer column is larger than that of the inner column. This difference occurs because the stiffness on the outer column side is smaller than that of inner column side.

Build.No.43-30F[Moment of Beams(tm)] Temp.400 ℃   Build.No.43-30F[Moment of Columns(tm)] Temp.400 ℃
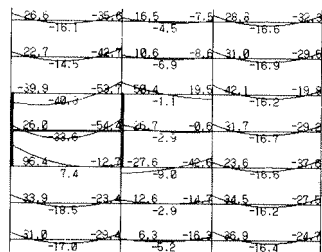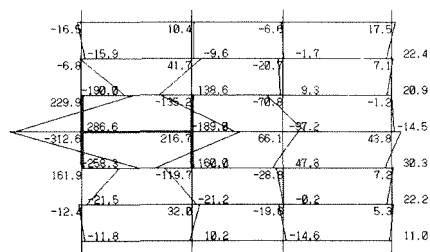
————   Local Substructure
———   Adjacent Substructure

Fig. 3   Moment Distribution in the Local and Adjacent Substructure

Beam Center

Mt,Nt: Bending Moment and Axial Force
      as Resultant of Thermal Stress
My,t,Ny,t: Yielding Moment and
           Axial Force
           at Elevated Temperature
Mp,t: Plastic Moment
      under Axial Force

Outer Column Top

Mp,t/My,0

Mt/My,0

My,t/My,0

Nt/Ny,0

Mp,t/My,0

Ny,t/Ny,0

My,t/My,0

Nt/Ny,0

Mt/My,0

$My,t = Z(\sigma_{y,t} - Nt/A)$
$Ny,t = A\sigma_{Y,t}$
$Mp,t = \int y\sigma_{p,t} \times YdA$

$\sigma_{y,t}$: Yielding Stress
$y\sigma_{p,t}$: Ultimate Stress
        Distribution

2.0
1.5
1.0
0.5
0   100 200 300 400 500 600   Temp.(°C)

2.0
1.5
1.0
0.5
0   100 200 300 400 500 600   Temp.(°C)

Fig.4 Thermal Stress Development

Displacement (mm)

Outer Column Top
Inner Column Top
Beam Center

100
0
-100
-200
0 100 200 300 400 500 600
            Temp.(°C)

Fig.5 Deformation

Temp.(°C)

7 6 3 4   5   2   1

600
500
400
300
200
100
0
   0   5.0   10.0   15.0   20.0   25.0
                                          K/Ko

3 4 5
1   6
2 Fire 7
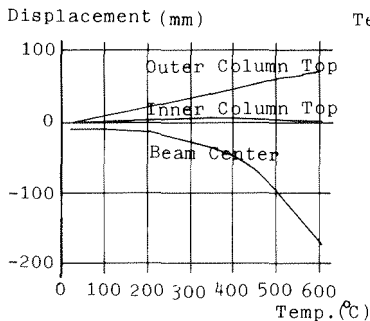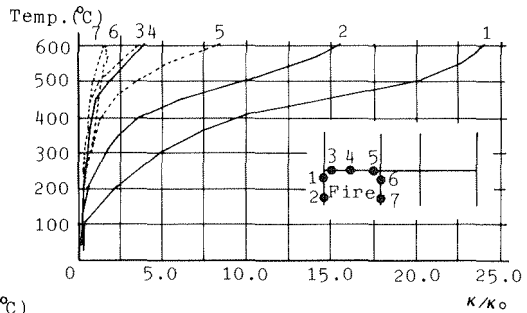
Fig. 6 Curvature

The deflection of the beam begins its rapid increase beyond 300°C. Fig.6 shows the curvatures at indicated points of the fire exposed beam and columns. At 600°C, the curvatures at the top and bottom of the outer column amount to 15 ∿ 20 times of the yielding curvature. At the center and both ends of the beam, the corresponding value is 4 ∿ 9 times of the yielding curvature.

4. SUMMARY OF RESULTS

The summary of the numerical results from the analysis of 48 building structures are shown in Table 2.

Thermal Stress

Significant thermal stresses occur in the local substructure constituting the compartment directly exposed to fire. Especially, the bending moments increases to high values at the top of the outer column (inner ones in type B) which are directly affected by the thermal elongation of beam, exposed to fire. The bending moments reaches their maximum value around 400°C and then decreases following the reduction of

load bearing capacity. In general, the axial force practically remain constant in the outer column but increase a little in the inner column. Due to restraint on both sides, the axial force in the fire exposed beam increases remarkably. It reaches about 50% of yield axial force around 400°C and then decreases to nearly zero at 600°C. On the other hand the variation of the bending moment in this beam is relatively small.

Table 2 shows yield temperature at which thermal stress reaches the yield point stress at the top of outer column and center of beam exposed to fire. The yield temperature at the top of the outer column is 115°C as minimum, 444°C as maximum and 189°C as average. The tendency is recognized in the yield temperature of the column where it becomes lower as the beam connected to the column becomes longer. Yield temperature in the beam is higher than that of the column, namely, 188°C as minimum, 568°C as maximum and 388°C as average.

Curvature

In some of the analyzed building structures, plastic rotations are noticeable at the top and bottom of the outer columns and at the center and the ends of the beam. Some of the maximum curvatures at these sections amounts to 50 times the yield curvature at room temperature. The temperatures at which the curvature exceeds 10 times the yield curvature at room temperature are shown in Table 2 as "Plastic hinge temperature". According to this definition, plastic hinges occur at 428°C and 501°C in average on the top and bottom of outer column, respectively. There are 15 building strucrures in which plastic hinge occurred on beams. In 7 buildings, namely, No.12,17,19,23,28,29,49, three plastic hinges occur at the center and the ends of the beam, i.e. a collapse mechanism is developed.

Deflection

There are several test standard criteria for a definition of structural failure or instability of columns and beams, exposed to fire. In this paper, deflection criteria used are $L^2/800H$ for beams and $h/30$ for columns. L, H and h are beam length, beam depth and column height, respectively. Horizontal deflection of column top and vertical deflection of beam center at 600°C divided by these criteria are shown in Table 2 as "Maximum deflection". Maximum deflection ratio(=calculated deflection / deflection according to criterion) for column is found for building structure type C,No.39, amounting to 65%. There is a tendency that longer span beams have larger deflection ratio. Building structures in which beam deflection ratio exceeds 80% are No.12,23,28 of type A and No.17,19,29 of type B. Beam collapse mechanisms are then developed for all these buildings.

5. CONCLUSION

(1) It is sufficiently safe when a high rise steel structure is designed according to existing Japanese fire resistance criteria (below av.350°C and max.450°C) because necessary load bearing capacity is maintained though yield zones and plastic hinges may appear before the criteria are reached.
(2) Some of long span beams will get collapse mechanisms if they are designed according to allowable temperature 600°C which is permitted in some countries.
(3) It is verified that the suggested calculation method based on division of structure into three types of substructures is suitable for high rise building structures exposed to compartment fire.

Table 1. List of High Rise Buildings Investigated

Types	Building No.	Number of Stories	Floor Height (m)	Span Direction		Length Direction		Floor in Fire
				Span	Length (m)	Span	Length (m)	
A	01	26	3.65	3	12.30	7	6.00	15
	08	32	3.95	3	16.50	9	6.00	18
	10	32	3.55	3	4.71	10	6.10	20
	12	14	3.70	3	19.20	7	6.20	7
	16	30	3.74	3	12.80	9	6.20	15
	18	15	3.80	3	13.20	8	6.25	8
	20	15	3.60	3	10.80	6	5.70	9
	21	22	3.70	3	12.80	6	6.00	12
	23	32	3.60	3	11.55	5	9.00	18
	25	19	3.55	3	10.40	6	6.40	10
	28	30	3.80	3	15.00	7	6.00	14
	31	34	3.70	3	14.70	18	3.00	17
	32	25	3.70	3	11.90	10	6.00	14
	33	31	3.80	3	12.40	11	6.20	16
	38	16	3.90	3	12.70	11	6.50	8
	40	43	4.00	3	13.00	18	3.00	30
	41	55	3.68	3	15.60	18	3.20	28
	42	55	3.65	3	15.40	19	3.00	29
	43	60	3.70	3	15.60	21	3.20	30
	44	37	2.95	3	8.80	7	7.40	21
	45	21	3.65	3	11.40	5	6.00	11
	46	18	3.90	3	12.00	11	6.00	11
B	17	19	3.88	3	22.00	12	3.20	10
	19	15	3.85	3	18.80	3	7.20	7
	29	14	4.20	3	22.00	7	6.26	8
C	26	18	3.10	3	5.20	4	8.55	12
	27	14	3.80	3	8.00	10	5.70	9
	34	23	3.80	3	12.40	8	6.20	12
	35	24	3.80	3	12.00	5	6.00	13
	39	19	3.45	3	11.45	6	6.40	9
	47	13	3.75	3	12.00	9	9.00	6
	49	20	3.52	3	10.40	6	6.40	11
D	39	19	3.45	3	11.45	6	6.40	9
	47	13	3.75	3	12.00	9	9.00	6
E	05	15	3.49	2	9.60	7	7.00	8
	09	17	3.50	2	8.00	5	6.40	10
	22	14	3.50	2	12.00	4	6.00	7
	48	16	3.70	2	13.65	12	6.40	8
F	05	15	3.49	2	9.60	7	7.00	8
	09	17	3.50	2	8.00	5	6.40	10
	13	15	3.35	2	12.00	12	6.00	10
	22	14	3.50	2	12.00	4	6.00	7
THE OTHERS	02	20	3.70	3	14.25	4	9.50	10
	03	17	3.20	1	7.55	4	7.00	9
	04	16	3.82	1	22.40	18	3.20	10
	06	27	3.20	6	5.50	10	15.00	24
	07	32	4.80	11	7.20	15	3.60	16
	11	25	3.25	5	7.75	5	7.75	14
	14	30	3.65	1	12.95	15	3.20	18
	15	22	3.75	5	16.80	22	3.20	13
	24	25	3.55	3	11.00	5	11.00	14
	30	21	3.65	5	12.64	12	3.15	13
	50	35	3.30	5	4.25	5	8.00	20

A type　B type　C type　D type　Etype　Ftype　Build.No.14　Build.No.15

Build.No.2,11,50　Build.No.3　Build.No.4　Build.No.24

Build.No.30　Build.No.3　Build.No.7
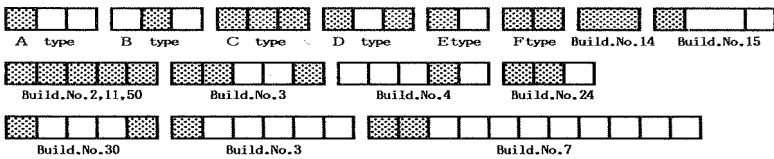
Table 2. The Results of Numerical Analysis

Types	Building No.	Service Load Rate Axial N. Inner Column	Service Load Rate Bend. M. Beam Center	Yielding Temp. (°C) Outer Column	Yielding Temp. (°C) Beam Center	Plastic Hinge Temperature (°C) Outer Column Top	Plastic Hinge Temperature (°C) Outer Column Bottom	Plastic Hinge Temperature (°C) Beam Left	Plastic Hinge Temperature (°C) Beam Cent.	Plastic Hinge Temperature (°C) Beam Right	Maximum Deflection (600°C) Beam Center	Maximum Deflection (600°C) Outer Column
A	01	0.14	0.08	174	512	385	460	—	—	—	0.276	0.496
	08	0.28	0.26	183	417	430	—	—	—	—	0.535	0.320
	10	0.12	0.02	347	479	—	—	—	—	—	0.128	0.094
	12	0.18	0.34	115	272	375	490	500	520	450	1.051	0.555
	16	0.23	0.14	171	457	420	515	—	—	—	0.422	0.375
	18	0.09	0.10	186	500	405	480	—	—	—	0.297	0.468
	20	0.14	0.10	189	586	440	500	—	—	—	0.180	0.521
	21	0.16	0.16	194	477	440	515	—	—	—	0.338	0.458
	23	0.24	0.30	183	300	—	—	540	590	515	0.873	0.158
	25	0.12	0.04	273	426	—	—	—	—	—	0.106	0.401
	28	0.47	0.42	142	258	455	—	515	545	490	1.254	0.442
	31	0.30	0.23	154	300	445	590	600	—	550	0.555	0.434
	32	0.14	0.10	200	443	460	—	—	—	—	0.382	0.332
	33	0.25	0.20	143	414	405	475	—	—	—	0.321	0.591
	38	0.14	0.18	191	426	445	575	—	—	590	0.412	0.404
	40	0.14	0.12	116	500	475	—	—	—	—	0.304	0.159
	41	0.38	0.27	140	340	420	510	—	—	580	0.493	0.635
	42	0.30	0.17	134	454	400	480	—	—	—	0.420	0.474
	43	0.28	0.19	146	400	405	500	—	—	—	0.440	0.585
	44	0.15	0.23	263	386	—	—	455	—	455	0.696	0.128
	45	0.10	0.07	192	500	415	500	—	—	—	0.220	0.465
	46	0.16	0.14	184	403	430	490	—	—	—	0.139	0.556
B	17	0.11	0.42	200	168	—	—	400	480	400	1.206	0.109
	19	0.11	0.24	197	317	—	—	550	560	550	0.821	0.122
	29	0.12	0.23	217	240	—	—	485	560	485	1.023	0.045
C	26	0.07 0.07	0.03 0.03 0.03	220 226	416 465 425	480 490	—	—	—	—	0.218 0.055 0.185	0.228 0.212
	27	0.06 0.06	0.05 0.01 0.09	197 176	578 468 441	450 430	540 540	—	—	—	0.144 0.064 0.290	0.470 0.248
	34	0.17 0.17	0.15 0.04 0.15	163 163	400 411 400	405 405	—	590 575	—	575 590	0.862 0.391 0.862	0.320 0.320
	35	0.13 0.13	0.15 0.04 0.15	164 164	365 363 365	405 405	550 550	—	—	—	0.539 0.539 0.539	0.317 0.317
	39	0.15 0.15	0.15 0.07 0.15	168 168	431 400 431	330 330	425 425	—	—	—	0.291 0.384 0.291	0.652 0.652
	47	0.04 0.04	0.08 0.02 0.08	157 157	517 441 517	330 330	430 430	—	—	—	0.264 0.213 0.264	0.538 0.538
	49	0.20 0.20	0.27 0.29 0.27	118 118	472 355 472	380 380	500 500	487 433 413	520	413 433 487	0.588 0.724 0.588	0.401 0.401
D	39	0.15 0.15	0.15 0.07	191 191	453	390 390	475 475	—	—	—	0.276 0.276	0.557 0.557
	47	0.04 0.04	0.08 0.02	174 174	530 530	370 370	450 450	—	—	—	0.266 0.266	0.503 0.503
E	05	0.12	0.10	246	457	490	—	—	—	—	0.442	0.175
	09	0.09	0.06	254	569	460	550	—	—	—	0.192	0.334
	22	0.12	0.16	180	449	405	495	—	—	—	0.480	0.347
	48	0.14	0.12	194	440	430	—	—	—	—	0.505	0.280
F	05	0.12	0.10 0.10	163 169	421 421	450 450	—	—	—	—	0.476 0.483	0.242 0.216
	09	0.09	0.06 0.06	194 194	550 550	440 440	520 520	—	—	—	0.219 0.219	0.361 0.361
	13	0.14	0.13 0.13	177 177	458 456	600 600	600	—	—	—	0.444 0.449	0.377 0.313
	22	0.12	0.16 0.06	174 157	544 416	400 400	480 460	—	—	595	0.211 0.529	0.473 0.374
THE OTHERS	02	0.12 0.12 0.12	0.17 0.01 0.17 0.01 0.17	188 188	260 250 290 250 260	460 460	—	—	—	—	0.403 0.294 0.350 0.294 0.403	0.273 0.273
	03	0.06 0.06 0.15	0.04 0.04 0.02	444 326	350 500 220	—	—	—	—	—	0.037 0.044	0.150
	04	0.10	0.20	170	410	420	475	580	—	580	0.581	0.353
	06	0.02	0.02	380	—	600	—	—	—	—	0.313	0.175
	07	0.08	0.12	230	590	520	580	—	—	—	0.116 0.232	0.469
	11	0.22 0.24 0.24 0.22	0.09 0.09 0.09 0.09	152 152	590 275 300 500	410 410	—	—	—	—	0.201 0.601 0.601 0.201	0.426 0.426
	14	0.11	0.17	304	280	—	—	545	—	545	0.732	0.072
	15	0.12	0.17	207	530	455	500	—	—	—	0.331	0.251
	24	0.18 0.16	0.08 0.06	140	490 410	315	420	—	—	—	0.289 0.476	0.530
	30	0.14 0.14	0.26 0.26	170 170	300 300	385 385	465 465	—	—	—	0.321 0.321	0.544 0.544
	50	0.10 0.12 0.12 0.10	0.03 0.03 0.03 0.03	185 185	440 300 300 440	600 600	—	—	—	—	0.259 0.257 0.257 0.259	0.196

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